Corrugated Bars

for

Reinforced Concrete

1906

D. E. GARRISON,
President.

D. E. GARRISON, Jr.,
Vice-Pres. and Treas.

W. H. KENNEDY,
Secretary.

A. L. JOHNSON, M. Am. Soc. C. E.
Consulting Engineer.

A. E. LINDAU, Assoc. M. Am. Soc. C. E.
Company Engineer.

R. L. MURPHY,
Contracting Engineer.

EXPANDED METAL AND CORRUGATED BAR CO.

SUITE 925 TO 936 FRISCO BUILDING

ST. LOUIS, U. S. A.

SOLE AGENTS FOR CORRUGATED BARS.

JOHNSON AND UNIVERSAL TYPES.

REPRESENTATIVES

H. C. MILLER & CO.,
1 Madison Ave., New York City.

EDWARD A. TUCKER, C. E.,
683 Atlantic Ave., Boston, Mass.

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306 Baronne St., New Orleans, La.

GUARANTEE CEMENT & STONE CO.,
221 South 5th St., Minneapolis, Minn.

OMAHA STRUCTURAL STEEL WORKS,
Omaha, Nebraska.

KANSAS CITY BRIDGE CO.,
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Boston Building, Denver, Colo.

WOOLSEY CROWE SUPPLY CO.,
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608 Crossley Building, San Francisco, Cal.

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708-9-10 I. W. Hellman Bldg.,
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COLONIAL TRADING CO. (For Panama.)
116 Broad St., New York City.

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R. P. CAMDEN,
Apartado 2391, Mexico, D. F.

EASTERN ENG. & CONTRACTING,
13 Canton Road, Shanghai, China.

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INTRODUCTION

The notable feature of the recent development in reinforced concrete is the rapid advance and adoption of bars providing a mechanical bond between the metal and the concrete. The market is being flooded with all manner of devices calculated to prevent the slipping of the reinforcement. Plain bars are being provided with some form of anchorage, either at the end or at intervals along the bar, in the hopeless endeavor to make that class of material answer the purpose. Some plain bar advocates, realizing the futility of providing the reinforcement in a beam with anchorage at the ends only, are advancing the theory that a reinforced beam is not a beam properly speaking, but an arch, in which the reinforcement merely serves as a tie rod to take up the thrust of the arch.

This, however, only dodges the issue. There can no longer be any question that a reliable, continuous, mechanical bond is absolutely necessary to secure permanent and satisfactory results. And the predictions we have been making for several years, regarding mechanical bond, and the advisability of a high elastic limit, are now being verified.

ELASTIC LIMIT.—There has been a great deal of discussion as to the reliability of Considère's conclusions as to the ability of concrete to stretch without rupture ten or fifteen times as much when reinforced with metal as when unreinforced, and there is certainly reason to suspect that his results are incorrect, at least not true in all cases. Concretes will vary in stretchability, depending upon the materials used, and upon their wet or dry condition. Dry concrete is brash, much like dry timber, and laboratory results on bone-dry specimens would not be representative of open-air structures. Certainly the metal acts as an integrator, enabling us to obtain at all sections the maximum stretch of which each section is capable, instead of, at all sections, only that of which the weakest section is capable. This will give a proportionate elongation, according to the latest investigation, of from .0004 to .0005, equivalent to a stress in the metal of from 12,000 to 15,000 pounds per square inch.

All of this analysis, however, is really beside the mark. A stress in the imbedded metal of 50,000 pounds, if inside the elastic limit, can result in no harm to structures reinforced with such a material as the corrugated bar. In such a case, even if the cracks were as far apart as six inches, they would only have a width of .01″ and, at a depth of two or three inches below the surface, even if this crack extended clear down to the bar, as it might do if plain bars were used, it is doubtful if the mild acidity of the carbonic acid in

the air could corrode the metal between two such strongly alkaline surfaces only .01″ apart. However, this may be with the plain bar, it is certain that the crack could not extend down to the surface of a corrugated bar, as this would involve a slip along the bar, which would necessitate the shearing off of the concrete entering the recesses on the bar's surface, a condition only to be obtained with the demolition of the structure.

The true function of the metal therefore is not to *prevent* cracks, but to subdivide a given stretch into a great many cracks. If this is done, and a corrugated bar used, it is of no consequence when the cracking first begins, nor what the stress in the metal reinforcement is, so long as it is inside the elastic limit, be that limit however high.

Inside the elastic limit, then, we have no damage. Beyond this limit, however, we encounter cracks of very large extent, which would soon result in the collapse of the structure. Therefore in our judgment, the factor of safety for reinforced concrete should be based upon the capacity at the elastic limit of the metal reinforcement, and should be, generally speaking, not less than four.

The building laws of many cities which now allow a working stress in the metal reinforcement of 16,000 pounds per square inch, whatever kind of metal it may be, even though it has an elastic limit of not over 30,000 pounds per square inch, are examples of reckless disregard of the public safety.

If, therefore, we are safe inside any reasonable elastic limit, and our working stress is the limit divided by our factor of safety, which should be not less than four, then it is wise and economical to have as high an elastic limit as possible consistent with such ductility as may be required by the work in hand. Generally little ductility is needed, but in some cases where much cold bending has to be done, medium or even soft steel might be required, and all three grades we are prepared to furnish.

MECHANICAL BOND.—There are three influences affecting the adhesion of cement to a metal surface, as follows:

1°. Breuillié, at La Châinette, reported some investigations in Annals des Ponts et Chaussèes for 1900, which showed that soaking in water for nine months reduced the adhesion of concrete to metal from one-half to two-thirds.

2°. Prof. Schule, who now occupies the position at Zurich formerly held by Prof. Bauschinger, reported at the International Engineering Congress at St. Louis in October, 1904, that when the reinforcing bars were stressed, even though inside the elastic limit, the cross section was slightly reduced. Inasmuch as the adhesion consists, simply, in the entering by the cement particles into microscopical pores on the surface of the metal, any shrinkage of the cross section of the metal, however slight, was sufficient to materially affect the value of this adhesion.

3°. In our experience we have had cases of rupture of the adhesion with plain bars after eight years' use, where the structure was not wet, nor did the stress in the bars ordinarily amount to much, this failure being due entirely to vibrations and shocks.

In Fig. on p. 14 is shown the photograph of the underside of a warehouse floor of concrete reinforced with plain bars, showing numerous cracks in the ribs, and about $\frac{3}{4}''$ deflection in a span of 8′. The floors were tested when put in to 800 pounds per square foot with very slight deflection, but after eight years use much of the floor, where much handling of goods took place, failed on account of the loosening of the grip of the concrete on the bars, and had to be replaced. The photograph shown is the underside of the new floor of same style and materials after four years use.

In open-air structures all three of these influences will generally be found working at the same time. Starting with 500 pounds per square inch adhesion, suppose only one-half this is lost by being wet much of the time, this leaves 250 pounds. If one-half of this is lost by shrinkage of the cross section of the metal, due to stress in same, we then have only 125 pounds. Taking a factor of safety of four, and making no allowance whatever for vibrations and shocks, which alone are sometimes sufficient to destroy the whole of the

adhesion, we have an allowable working stress for adhesion of 30 pounds per square inch. For a rod of 1″ diameter this means about 1200 pounds per lineal foot, which, to develop a working stress in the metal of 12,000 pounds per square inch, would require an anchorage of ten feet in which no other increment could be added! Such a requirement in practice would be absurd and impossible, generally speaking.

That foreign engineers, who have been mainly responsible for the use of plain bars for concrete reinforcement, are coming to realize the unreliability of adhesion alone, is indicated in many ways, chief of which is that the specifications prepared about a year ago, covering all this kind of work in the German Empire, state that "the bond shall, so far as possible, be of a mechanical nature." Up to that time there had been practically nothing used but plain bars. Further, it is noticeable that most of the French companies are now turning up their rods at the end or using some similar device, though what advantage is to be gained by turning up a three-quarter inch rod sixteen feet long an inch or two at the end, it is hard to realize.

Foreign engineers, as a matter of fact, have not had the experience that we have. Their beam work, in which alone these weaknesses develop, dates back only eight or nine years, while in the United States we have been building beams almost continuously since 1875. As it has taken eight years for

this weakness to develop in some of our own work, and as abroad they first used mortar instead of concrete, which gives a stronger adhesion, it may be said that the time is only just arriving when we might expect them to discover the necessity of using other means of obtaining a reliable bond. And as before stated, these expectations are now realized.

When the unreliability of the adhesion is admitted, then it becomes necessary to have a mechanical bond that will avoid all splitting tendency on the concrete. This requires, with mathematical certainty, that the side of the ribs on the bar shall not vary from a plane at right angles to the axis of same by an amount greater than the angle of friction between the concrete and metal which is, generally speaking, about 45°. The corrugated bar is the only one in the market that fullfils, or that can ever fullfil, this condition, as our patent covers all bars that can be rolled in which the condition is complied with.

Summing up the situation, the corrugated bar has the following vital points of advantage over plain bars, and over all other types of bar reinforcement:

1°. Its elastic limit being high (unless by special requirement) enables a higher working stress to be used than should be used for soft steel bars, taking, therefore, proportionately less metal.

2°. Cracks in the concrete can not penetrate to the corrugated bar so long as the stress in the steel is inside the elastic limit.

3°. Soaking in water concrete reinforced with corrugated bars does not injure their bond.

4°. Reduction of the cross section of these bars, due to tension stress inside the elastic limit, in no way reduces their effective grip on the concrete.

5°. Vibrations and shocks do not impair their bonding value.

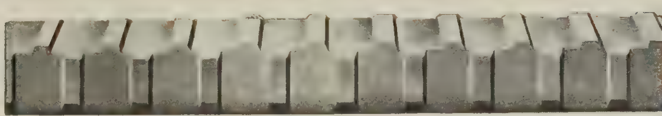
6°. Being formed by rolls while hot, the bars are all alike, the shape of each piece not depending upon the personal equation of some workman.

Attention is called to a new form of corrugated bar—the Universal type—which will be found useful wherever great flexibility is required.

The analysis of rectangular beam has been carefully remodeled and placed on, what is hoped, will be found a more general and rational basis. A discussion of circular and annular beams has been added, together with many new tables, illustrations and details of construction.

Underside of Warehouse Floor, Tamm Glue Co., Showing Cracks and Deterioration After Four Years' Service. Plain Bars Used. See page 10 of Introduction.

1/2" ☐ Bar. Net Section 0.18 ☐"; Weight 0.64 lbs. per ft.



3/4" ☐ Bar. Net Section 0.37 ☐"; Weight 1.35 lbs. per ft.

[illegible] Bar. Net Section [illegible]; Weight 1.95 lbs. per ft.

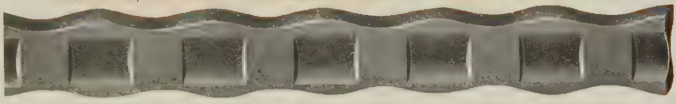
1" ☐ Bar. Net Section 0.70 ☐"; Weight 2.70 lbs. per ft.

1 1/4" ☐ Bar. Net Section 1.07 ☐"; Weight 4.00 lbs. per ft.

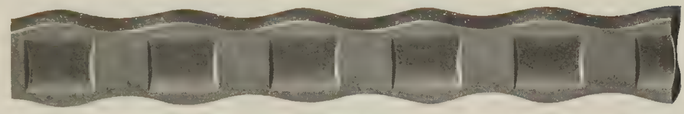
Old Style Corrugated Bars.
A variation in weight of 5% either way is required.

New Style Corrugated Bars.
A variation in weight of 5% either way is required.

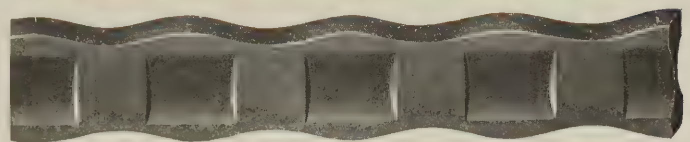
No. 1 [illegible] x1 Bar Net Section 0.19[illegible] : Weight 0.73 lbs. per ft.



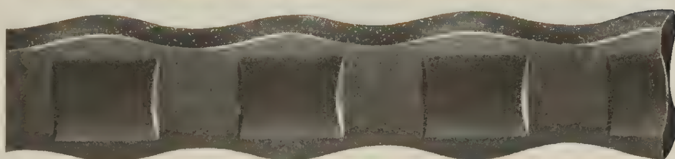
No. 2 [illegible] x1[illegible] Bar. Net Section 0.32 : Weight 1.18 lbs. per ft.



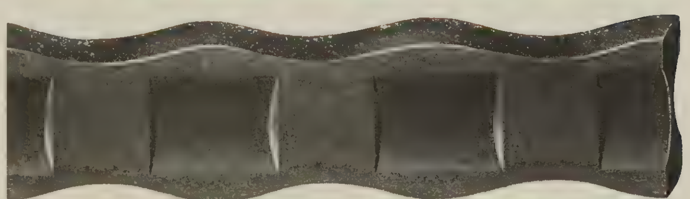
No. 3 [illegible] x1[illegible] Bar. Net Section 0.41 : Weight 1.35 lbs. per ft.



No. 4 [illegible] x1[illegible] Bar. Net Section 0.54 : Weight 1.97 lbs. per ft.



No. [illegible] x2 Bar Net Section 0.65 : Weight 2.27 lbs. per ft.



No. 6 [illegible] x2[illegible] Bar Net Section 0.80 : Weight 2.[illegible] lbs. per ft.

Universal Type Corrugated Bars. A variation in weight of 5% either way should be allowed for. Larger sections can be rolled if required.

BUILDINGS AND
BUILDING DETAIL

THE BUREAU OF BUILDINGS,

OFFICE OF SUPERINTENDENT

FOR THE BOROUGH OF MANHATTAN.

Nº 220 FOURTH AVENUE.

S.W. CORNER 18TH ST.

BY

The City of New York, Dec. 30, 1905.

Messrs. H. C. Miller & Co.,
1 Madison Av., City.

Gentlemen:-

As a result of the fire and water tests on Dec. 26th, 1905, under the supervision of this Bureau, your form of reinforced concrete construction, known as the Corrugated Bar System, is approved for general use in the Borough of Manhattan, as a fireproof construction.

This approval is issued in accordance with the Regulations of this Bureau, and on condition that such construction is made in accordance with these Regulations, and such construction and the strength of the same is determined in accordance with these rules and regulations;

Further, that all steel used in the construction shall be surrounded on all sides with at least one inch of concrete in the slab construction and at least one and one-half inches in the beam, girder and column construction;

THE BUREAU OF BUILDINGS,

OFFICE OF SUPERINTENDENT

FOR THE BOROUGH OF MANHATTAN.
Nº 220 FOURTH AVENUE.
S.W. CORNER 18TH ST.

2. H.C.M.& Co. The City of New York,

Further, that no column used in this construction shall be less than ten inches;

Further, that the minimum thickness of slab and floor construction shall be three and one-half inches.

Your reinforced cinder construction as tested is approved for general use in the Borough of Manhattan, as a fireproof floor construction, for spans up to eight feet and live loads of one hundred and fifty pounds per square foot, provided it is constructed as tested and in accordance with the specifications on file in this Bureau. A detail of the construction, as approved by this Bureau, is enclosed herewith.

Yours truly,

Isaac A Hopper

Superintendent

(Enclosure)

Department of Public Safety.

Bureau of Building Inspection

Rooms 313-315-317-319,

Director

Chief of Bureau

City Hall.

Philadelphia, Feb.6, 1906.

Messrs. H. C. Miller & Co.,
C/o Walter Loring Webb,
Phila.

Dear Sirs:-

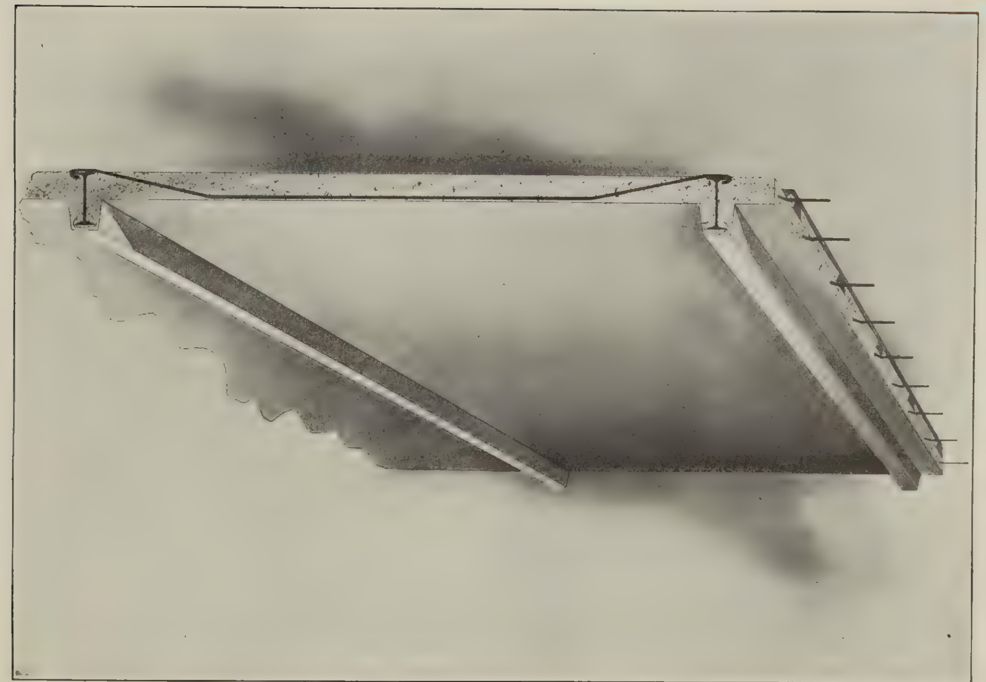
The fire and water test conducted by Prof. Francis C. Van Dyke, Ph.D., at New Brunswick, N. J.. on Dec. 26th, 1905, and witnessed by an inspector from this Bureau, is accepted by the same as a satisfactory reinforced concrete fireproof construction, and is approved for general use in the City of Phila.

This however is given upon the condition that all floors shall comply with the regulations of this Bureau and the construction and strength of same is to be determined in accordance with these Rules and Regulations.

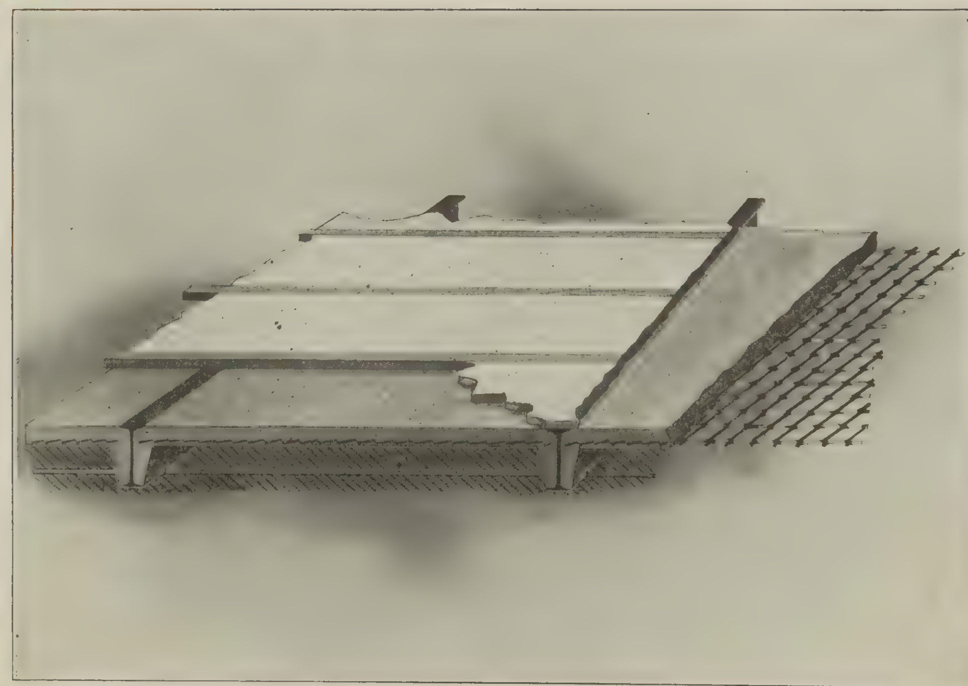
Yours truly

Edwin Clark

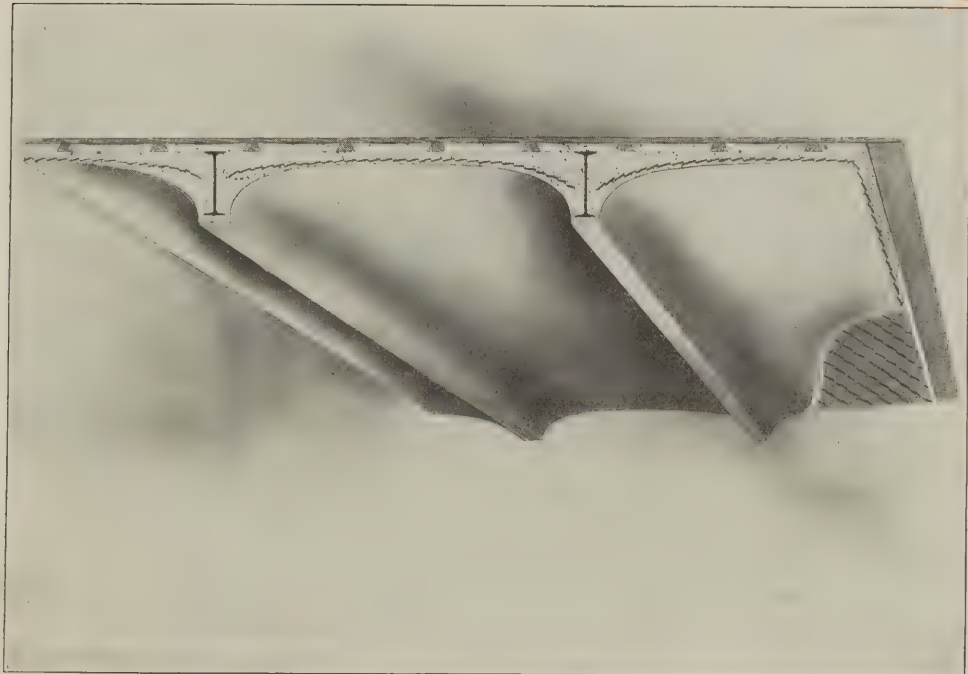
Chief



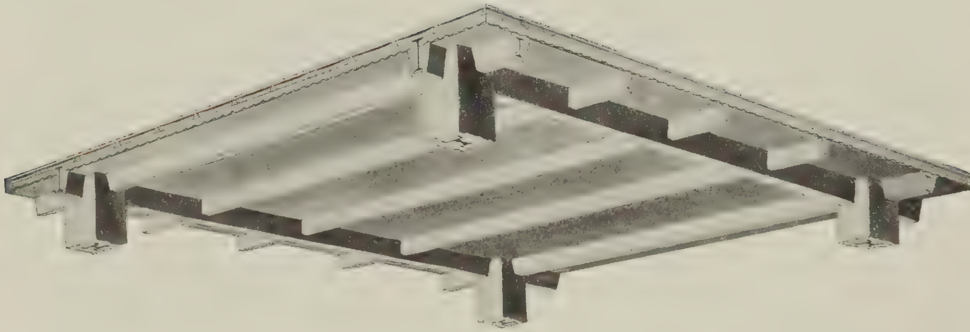
System No. 3.—Flat Slab Floor—For designing tables, see pages 200 and 203—Suitable for spans up to sixteen feet.



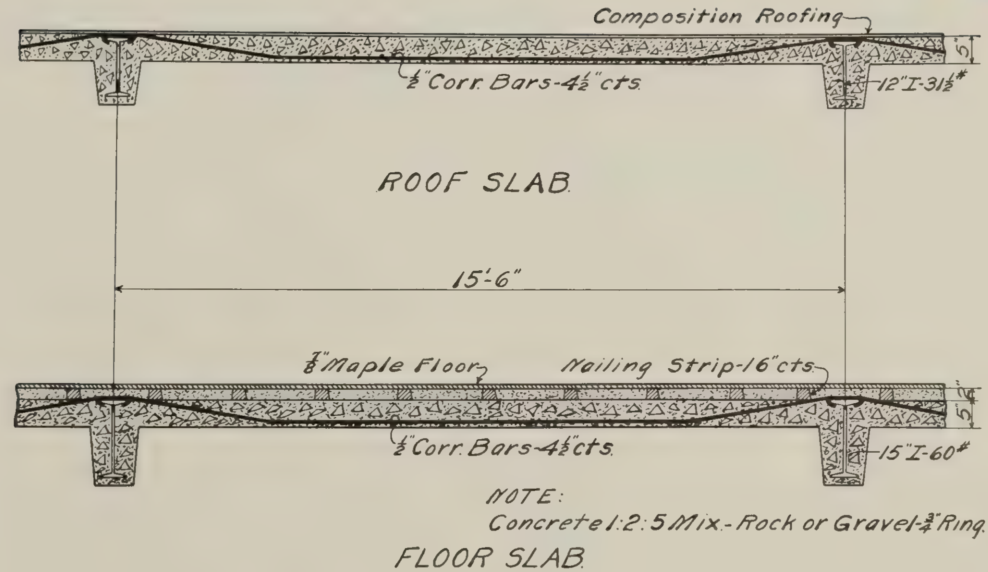
System No. 4—Expanded Metal Flat Slab—For designing tables, see pages 198, 199, 201 and 202. Suitable for spans up to eight feet.



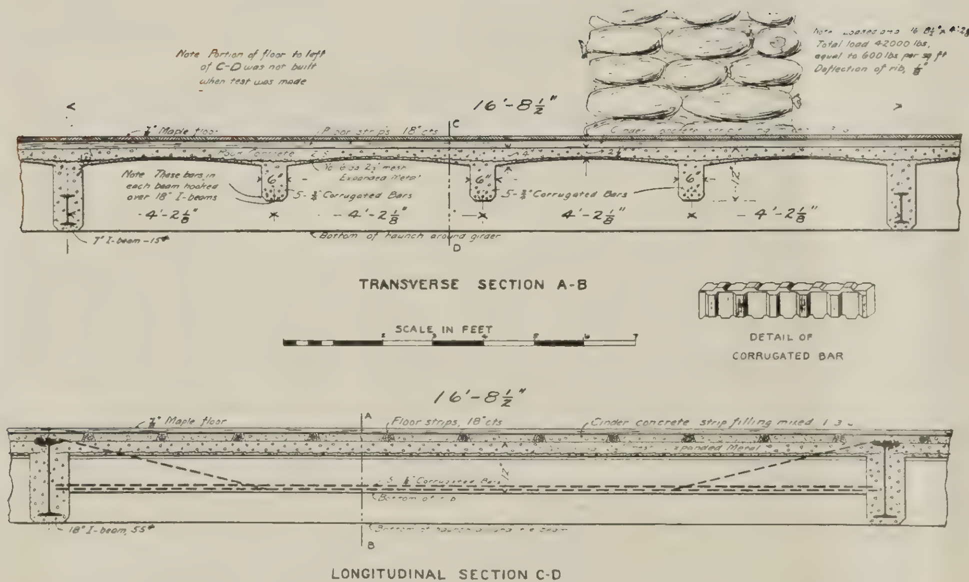
System No. 5.—Expanded Metal Flat Arch—Suitable for spans up to ten feet—No tie rods necessary.



System No. 6.—Long Span Tee System, Using Corrugated Bars in the Ribs and Expanded Metal in the Flat Slab. For designing table in good rock concrete, see page 194.

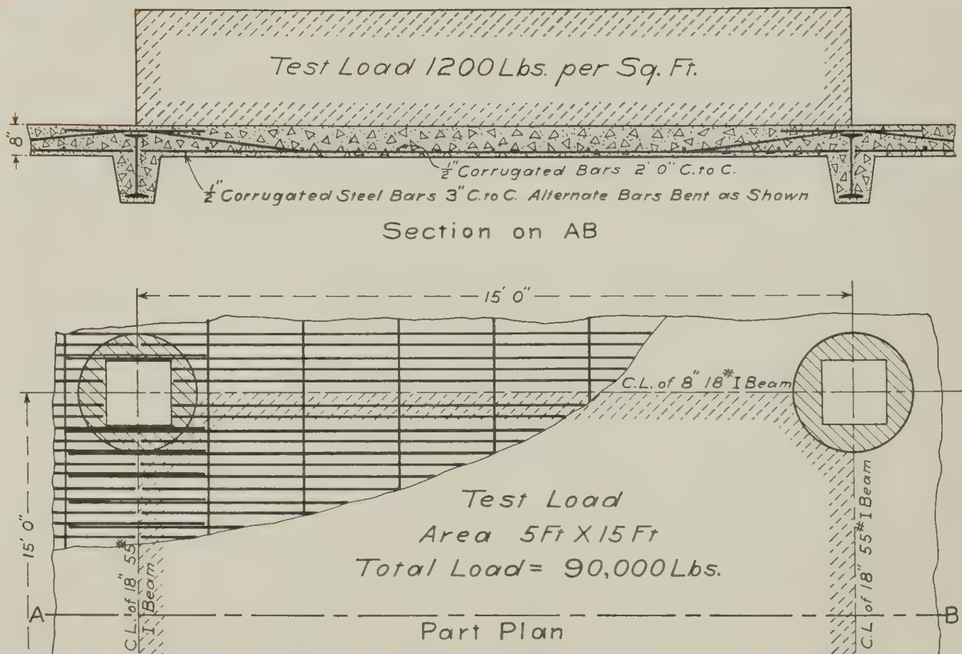


TYPICAL FLOOR AND ROOF SECTIONS.

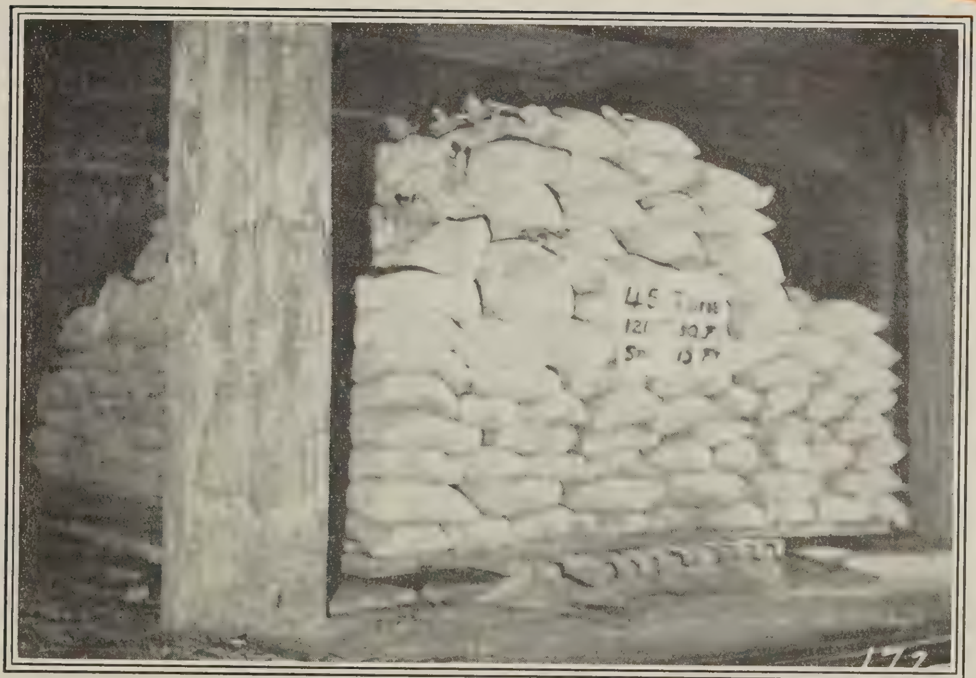


System No. 6.—Tee Floor—For designing table, see page 194.

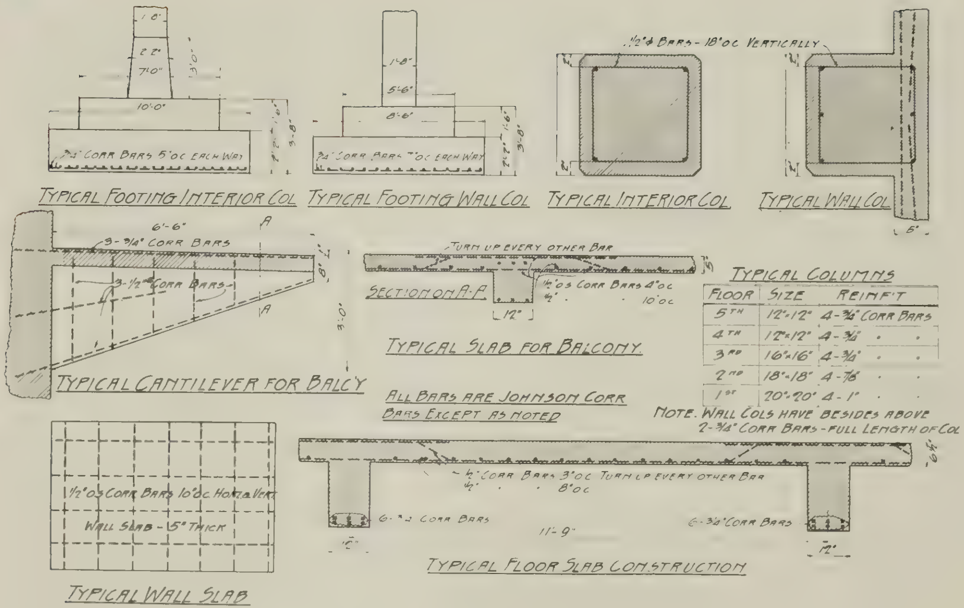
Test on System No. 6, as shown on page 28. Rock Concrete, 1:2:5; Age, 6 weeks. Load 600 pounds per square foot. Deflection at center of rib ⅛″.



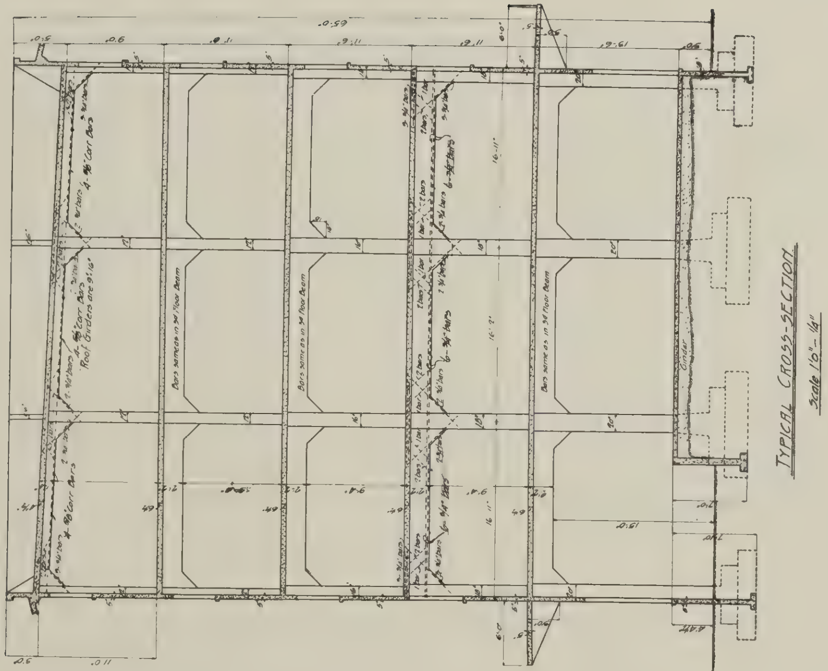
Test of Floor Construction, North American Cold Storage Building, Chicago.
Frank B. Abbott, Architect.
Hoeffer & Co., Contractors.



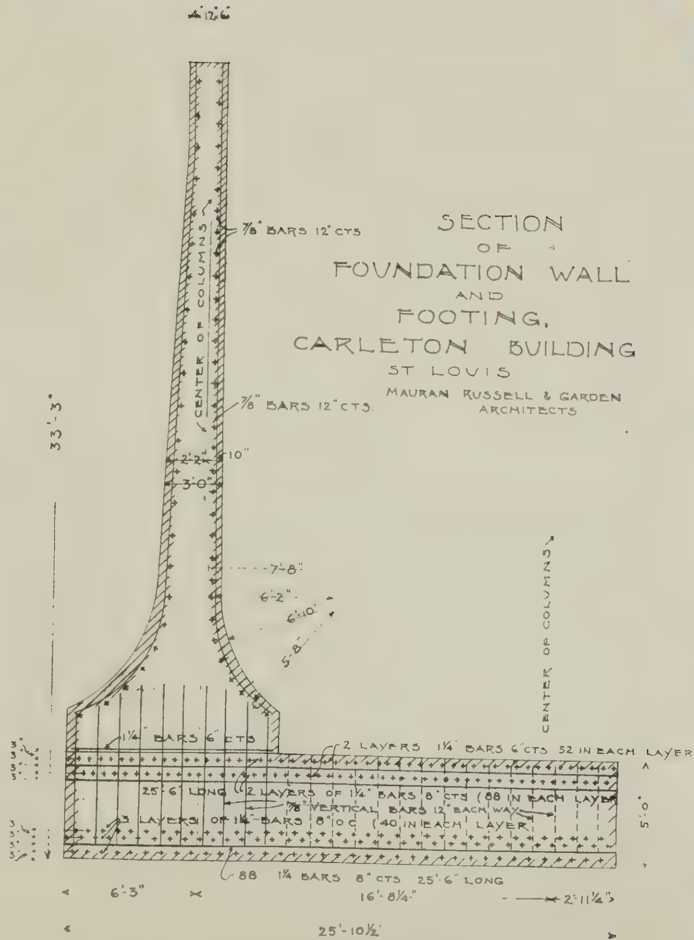
Test of Floor System, No. 3 (flat slab), North American Cold Storage Building.
Note probable absence of arching effect by use of this type of loading.



Typical Warehouse Details.



Typical Warehouse Details.

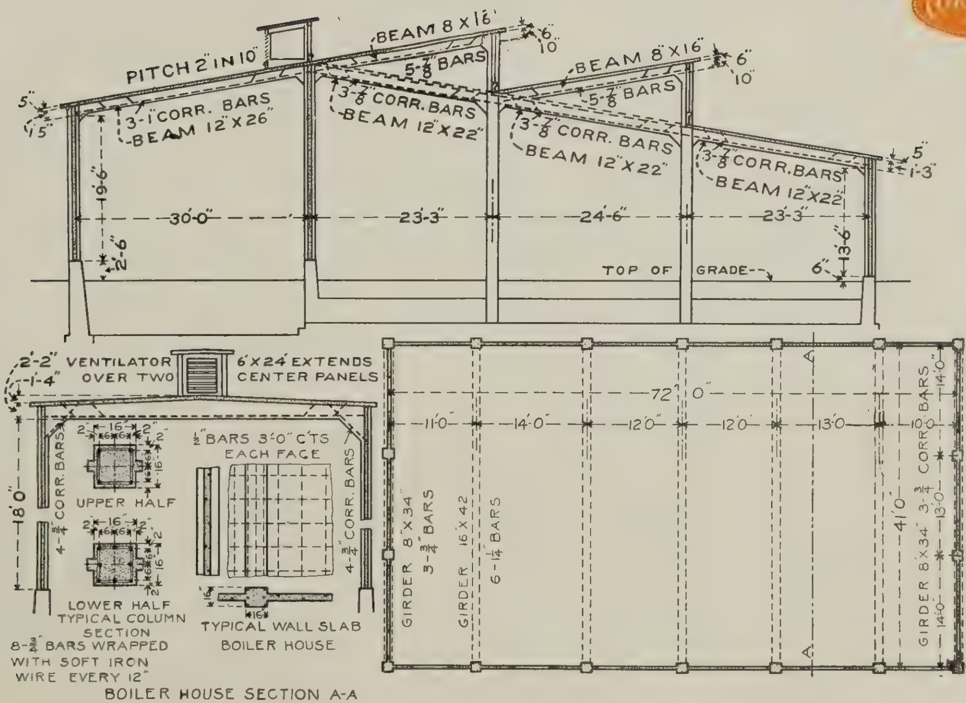


COLUMN FOOTINGS No 83-84-85-86-87-88

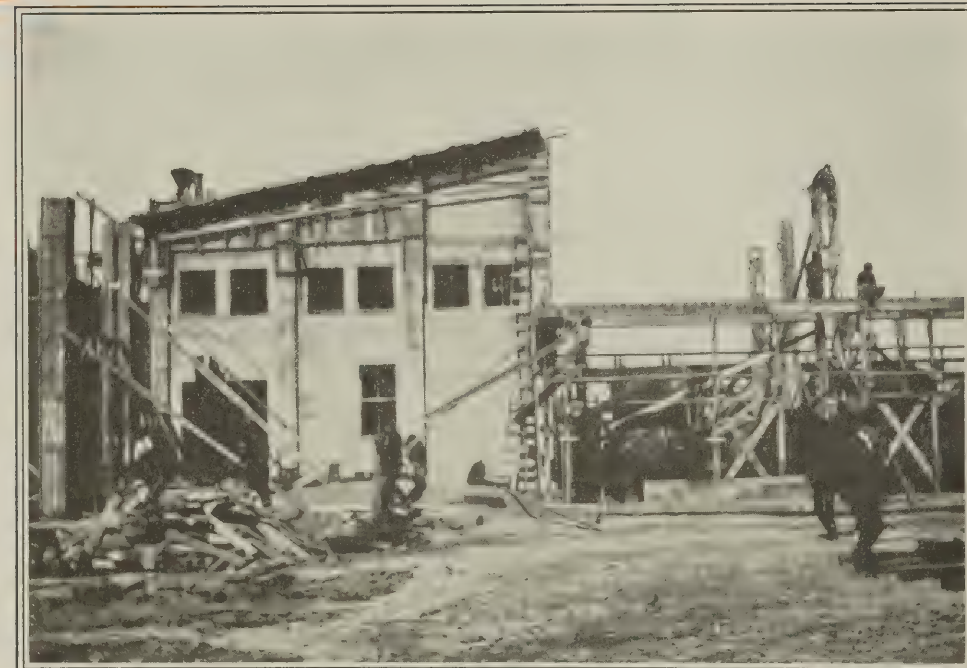
J. R. Broderick, Engr.

Carleton Building—Completed Retaining Wall.

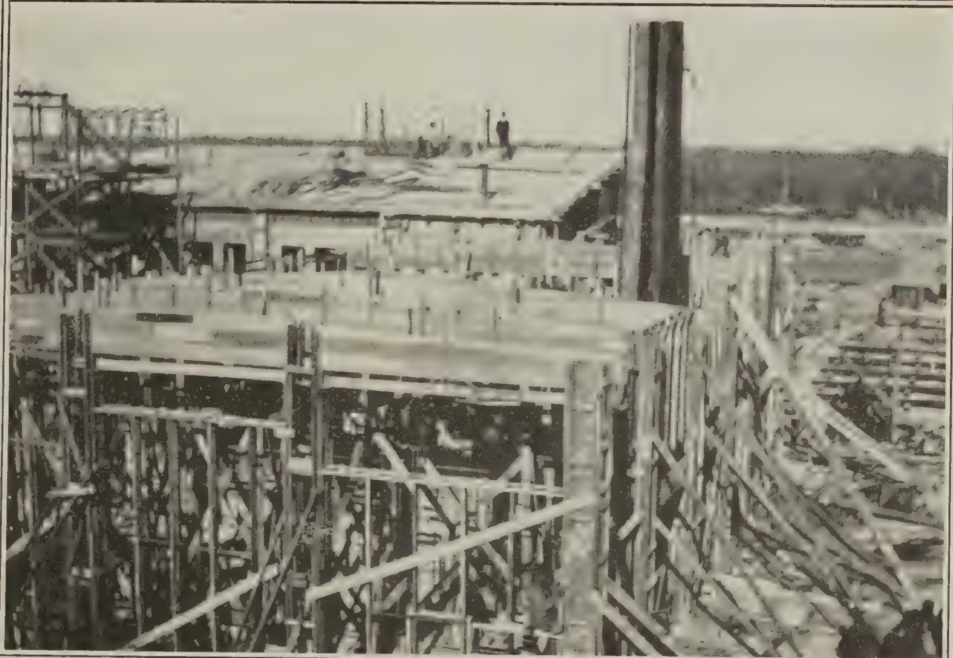
General View, Creosoting Plant, Somerville.



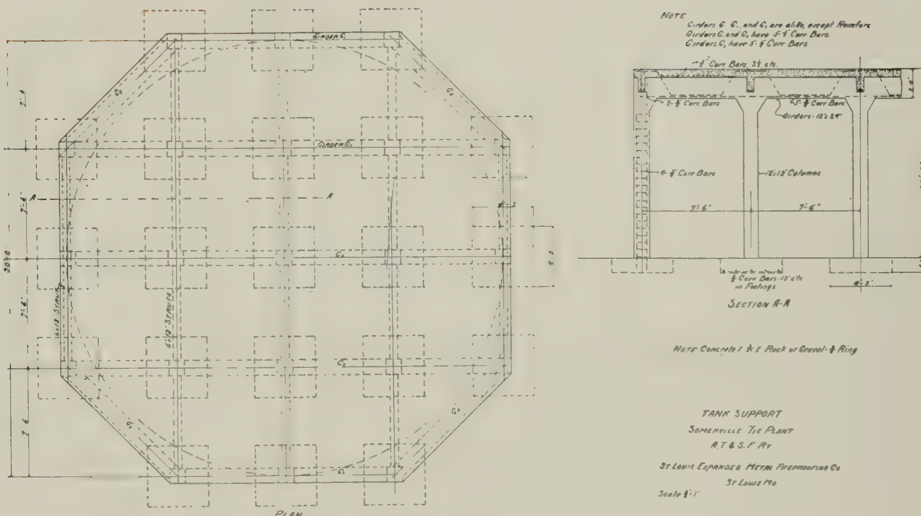
Cylinder and Boiler House, Creosoting Plant, Somerville, Tex.



Creosoting Plant. South Elevation. Pump Room. Cylinder House under Construction.

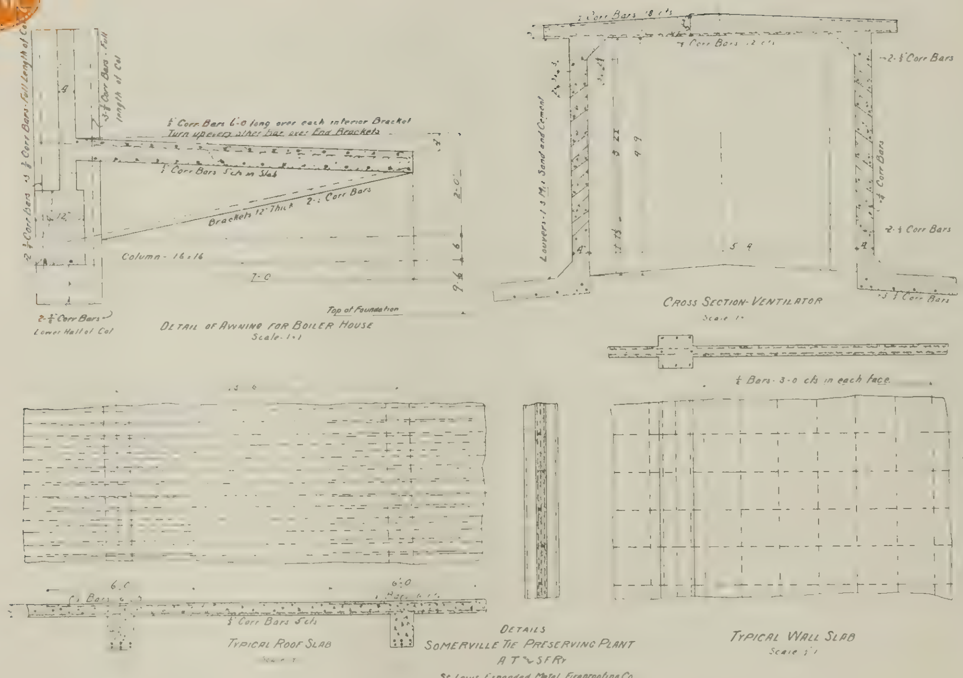


Creosoting Plant, Showing Boiler House Under Construction.



Tank Support, Somerville Tie Plant, A., T. & S. F.

Creosoting Plant, Somerville, Tex. Completed Tank Supports.



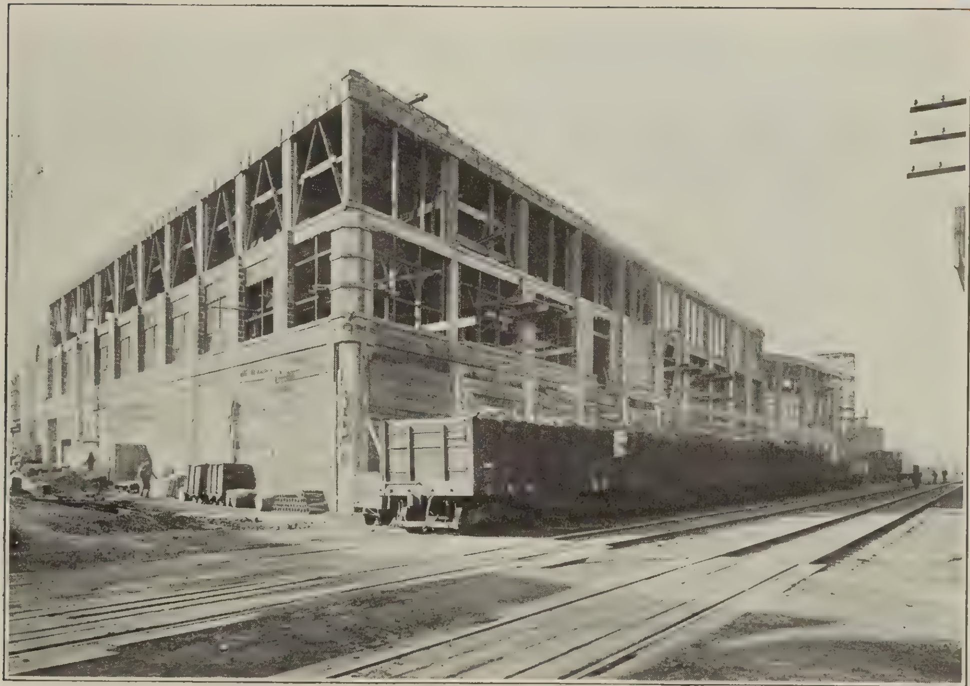
Creosoting Plant Details.

Keyser Office Building, Baltimore, Md.
Wyatt and Nolting, Archts.
Broderick and Wind, Contrs.
Basement floors and foundations designed to resist a 10-foot head of water.

Thompson and Norris Building, Brooklyn, N. Y.
Thompson and Norris Co., Owners and Builders.
Horace I. Moyer, Supt. in Charge Constr.
H. C. Miller & Co., Engineers.

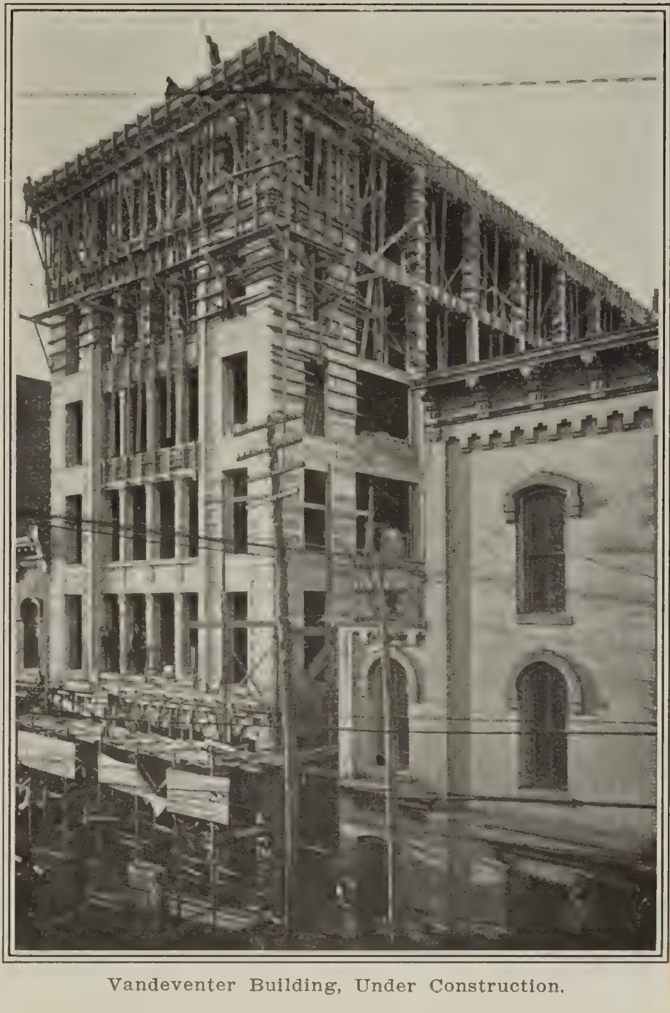
Thompson and Norris Building, Brooklyn, N. Y.

Dayton Malleable Iron Works.
Peters, Burns & Pretzinger, Architects.

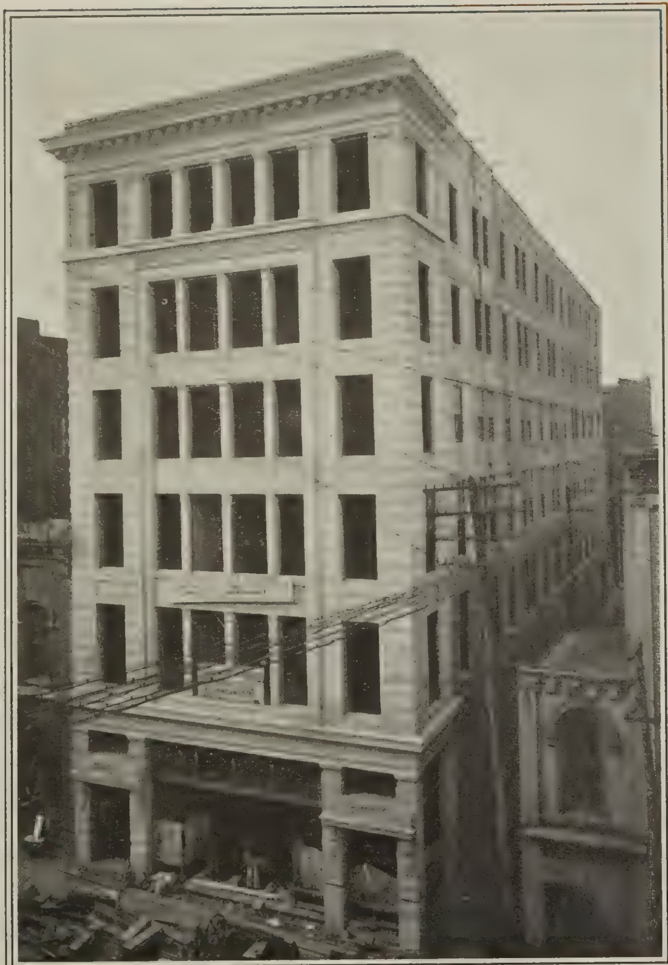


Dayton Malleable Iron Works.
Peters, Burns and Pretzinger, Architects.

Vandeventer Building, Knoxville, Floor Test.

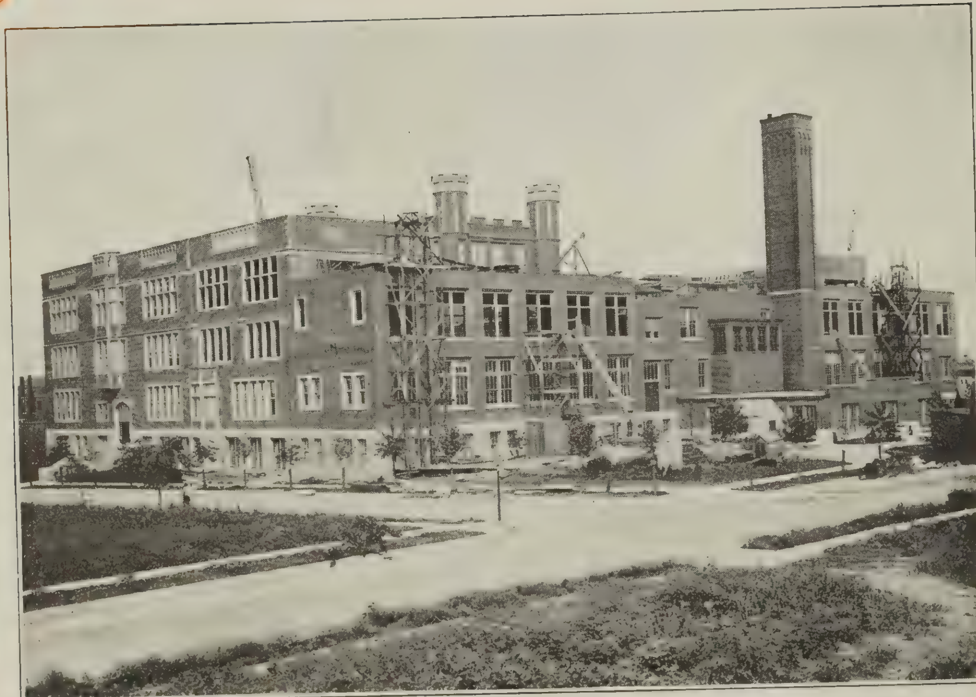


Vandeventer Building, Under Construction.



Vandeventer Building, Knoxville, Tenn.
Leon Beaver, Architect.
The Oliver Co., Contractors.

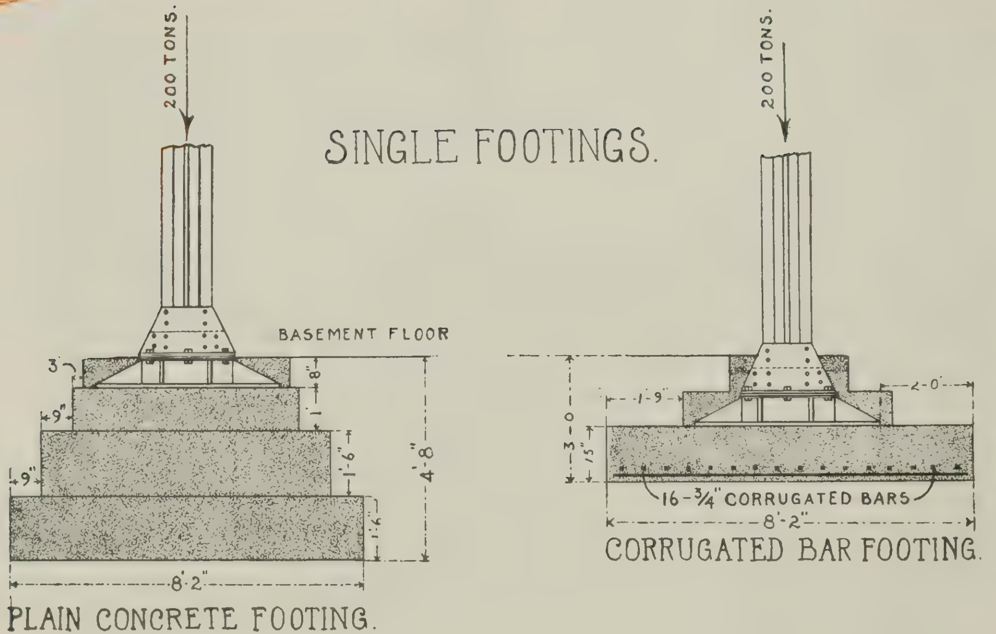
Wood Worsted Mills, Lawrence, Mass.
Dean and Main, Engineers.



Addition to McKinley High School, St. Louis.
Wm. B. Ittner, Com. School Buildings.

Addition to McKinley High School, Under Construction.

SINGLE FOOTINGS.



Comparison between Plain and Reinforced Single Footings.

COMPARISON OF COST OF SINGLE FOOTINGS

PLAIN CONCRETE FOOTING

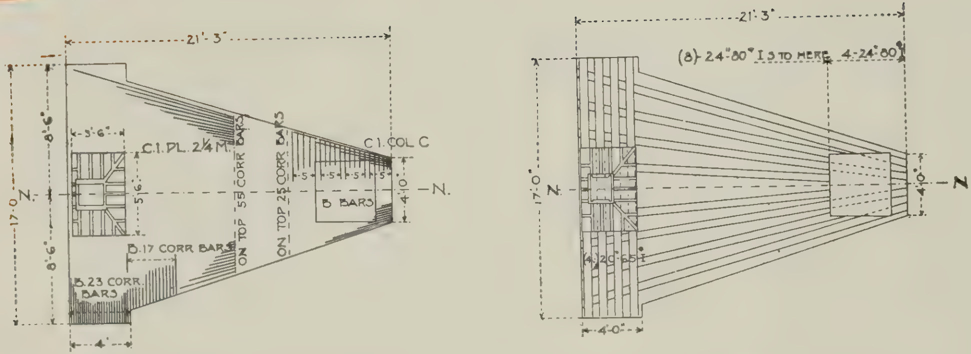
Excavation, 11½ cu. yds., @ 50c	$ 5.75
Concrete, 205 cu. ft., @ 20c	41.00
Total	$46.75

CORRUGATED BAR FOOTING

Excavation, 7½ cu. yds., @ 50c	$ 3.75
Concrete, 102 cu. ft., @ 20c	20.40
Corrugated Bars, 382 lbs., @ 2½c	9.55
Extra column length, 85 lbs., @ 3½c	2.98
Total	$36.68

This shows that even in single piers a distinct saving is made by the reinforced concrete design. The percentage of saving increases with the size of the footing.

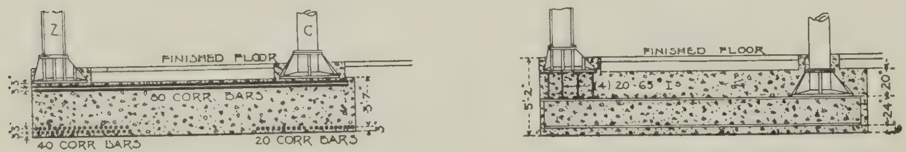
The chief recommendation of this construction, however, lies not so much in the decreased cost as in the greatly increased reliability. The plain footing depends upon the tensile strength of the concrete to give the required spread. No more unreliable factor of strength exists in the whole realm of building materials. In the corrugated bar design, even if the tensile strength of the concrete were zero, the strength of the footing would not be materially altered.



CORRUGATED · BAR DESIGN

STEEL · I · BEAM · DESIGN

DOUBLE · FOOTING



SECTION. N. N.

SECTION · N. N.

Comparison between Corrugated Bar and I Beam Double Footings.
Corrugated Bar Design used for the Norvell-Shapleigh Building, St. Louis.
Weber & Groves, Archts.

DOUBLE OR COMBINED FOOTINGS

On the foregoing page is shown a comparison between a Corrugated Bar and an I Beam footing, of equal strength, for two columns. The column to the left carries 358 tons, the other 222 tons. The area of the footing is 232 square feet, making an average pressure of 2.5 tons per square foot. The center of gravity of footing does not coincide with the resultant of the loads, resulting in a variation in soil pressure, which can be obtained by Hooke's law for beams $f = \frac{My_1}{I}$ where f is the increase or decrease in pressure in tons per square foot at the edge of the footing; y_1 the distance in feet from the edge in question to the center of gravity of footing; M is the revolving moment in foot tons around this center of gravity; and I is the moment of inertia of the footing plan in feet. In the case shown, I=7565. M=580x0.42=243.5 foot tons. From the small end to the center of gravity is 12.92′. This gives f_1=0.42 tons per square foot. In the same way f_2 is found to be 0.27 tons per square foot. Hence under one edge we have a pressure of 2.77 tons per square foot and under the other 2.08 tons.

The maximum bending moment occurs at the point of zero shear and is 22,800,000 inch pounds for a width of 11.77 feet. Taking a factor of safety of four, we have an ultimate moment for a width of 1′ of 7,760,000 inch pounds. From the beam tables, for 1:3:6 rock concrete, we get a required thickness of concrete of foot=7, o. s. corr. bars.

For the I-beam footing, the moment of 1,900,000 foot pounds requires 8, 24″—80 lb. beams.

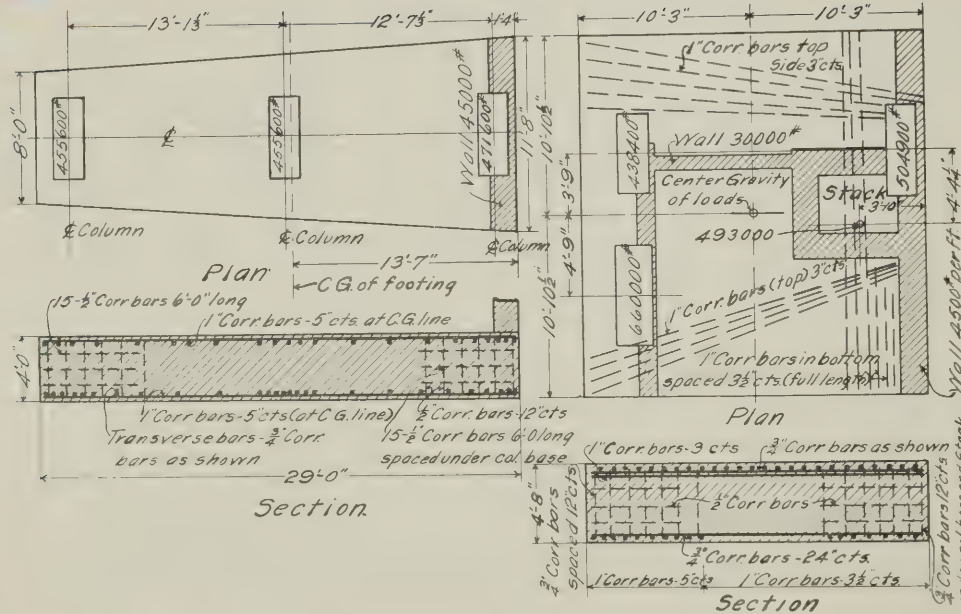
COMPARISON OF COST.

CORRUGATED BAR FOOTING.

Excavation, 39 cu. yds., @ 50c...	$ 19.50
Concrete, 870 cu. ft., @ 20c......	174.00
Bars, 4,106 lbs., @ 2½c.........	102.65
Total......................	$296.15

I-BEAM FOOTING.

Excavation, 45 cu. yds., @ 50c...	$ 22.50
Concrete, 966 cu. ft., @ 20c......	193.20
Steel Beams, 16,660 lbs., @ 2½c.	416.50
Bolts and sep's, 1,120 lbs., @ 2c..	22.40
Total......................	$654.60

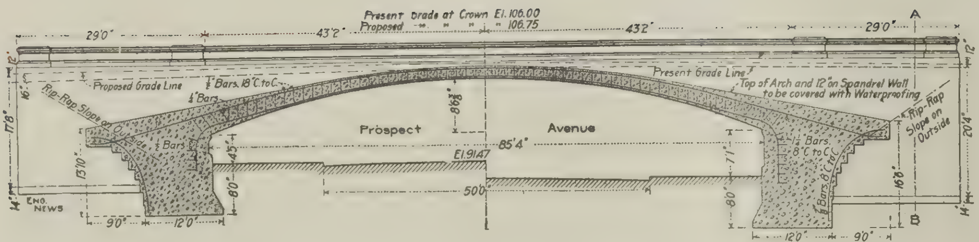


Typical Column Footings Installed in Blackstone Building, St. Louis.
H. F. Roach, Architect.

MISCELLANEOUS STRUCTURES

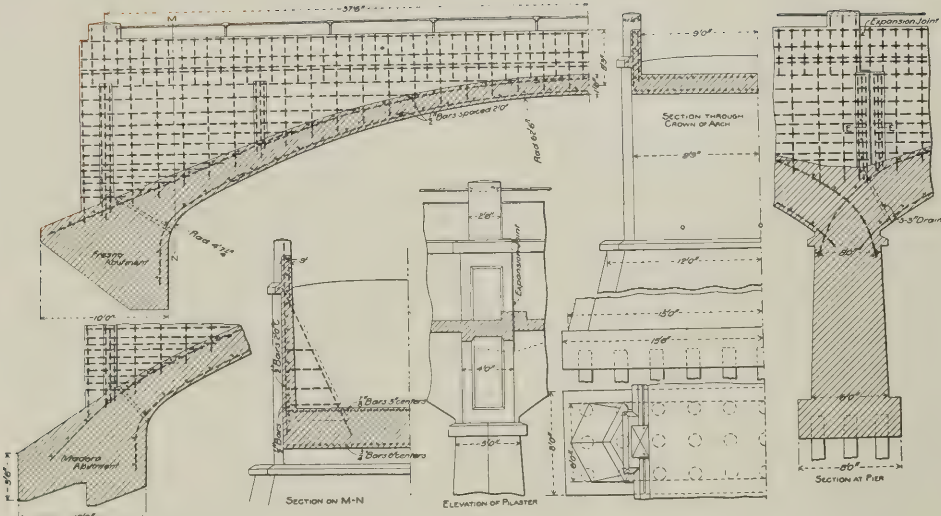
City Bridge, Reno, Nev., Two 65-foot Spans.
Designed by J. B. Leonard, C. E.
Built by Cotton Bros. & Co., Contrs.
T. K. Stewart, Engineer in Charge.

Completed Seeley Street Bridge, Brooklyn.



Seeley Street Bridge, Brooklyn, N. Y. G. W. Tillson, Chief Engineer.
E. J. Fort, Assistant Engineer.
D. Cuozzo & Bro., Contractors.

Seeley Street Bridge, Brooklyn, during Construction.

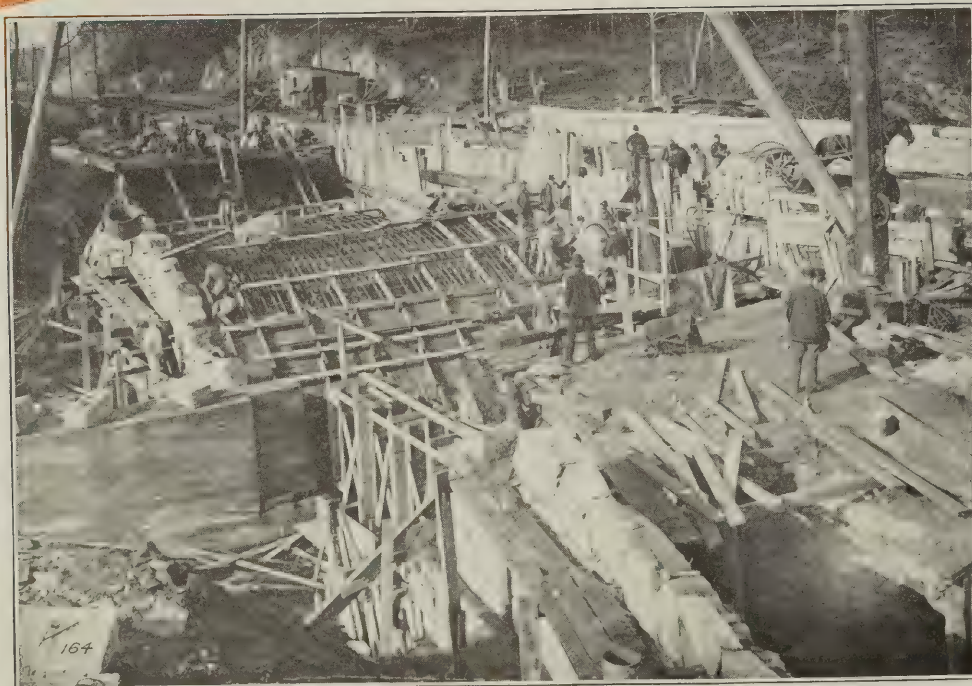


Reinforced Concrete Bridge, Pollasky, Cal., Ten 75-foot Spans.
Built by Pacific Construction Co.
Designed by J. B. Leonard.

Reinforced Concrete Bridge, Pollasky, Cal., Ten 75-foot Spans.
Built by Pacific Construction Co.
Designed by J. B. Leonard.

Dry Creek Bridge, Stanislaus Co. Span, 112 feet.
Designed by J. B. Leonard.

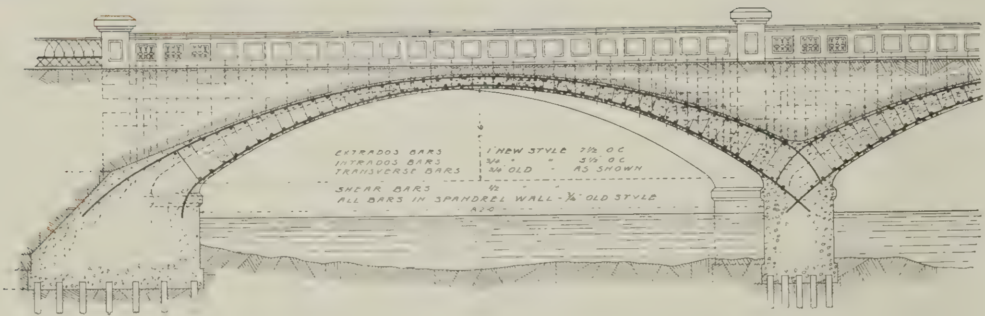
Elmwood Bridge, Memphis. Span, 100 feet.
J. A. Omberg, Jr., City Engineer, Memphis.

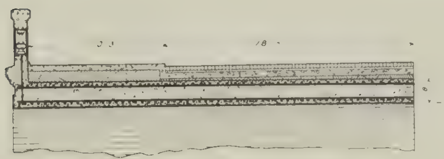


Bridge Over the Charles River at Newton Upper Falls, Metropolitan Park Commission, Commonwealth of Massachusetts.

J. R. Rablin, Engineer.

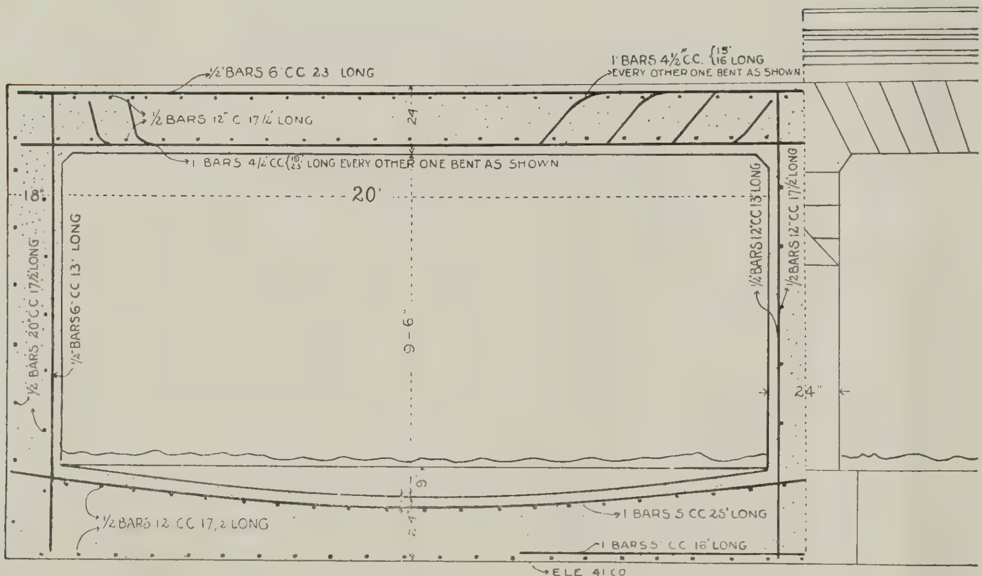
Two-Tail Race Arches, American Writing Paper Co., Holyoke, Mass.
Designed by Edward P. Butts, C. E.
Constructed by Caspar Ranger, Contractor.





-SECTION-OF-REINFORCED-
CONCRETE-ARCHES-AT-
-EAU-CLAIRE-WIS-

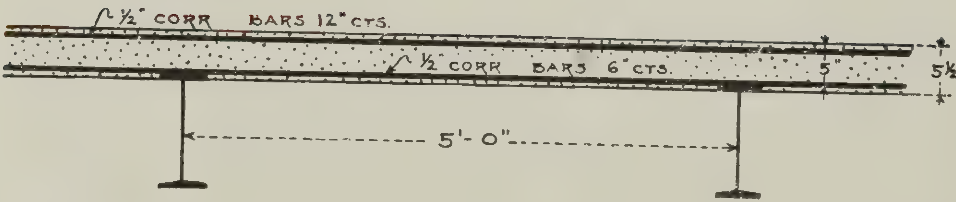
Bridge at Eau Claire, Two 82-foot Skew Spans.
McClellan Dodge, City Engineer.
Geo. Nelson, Contractor.



HALF SECTION

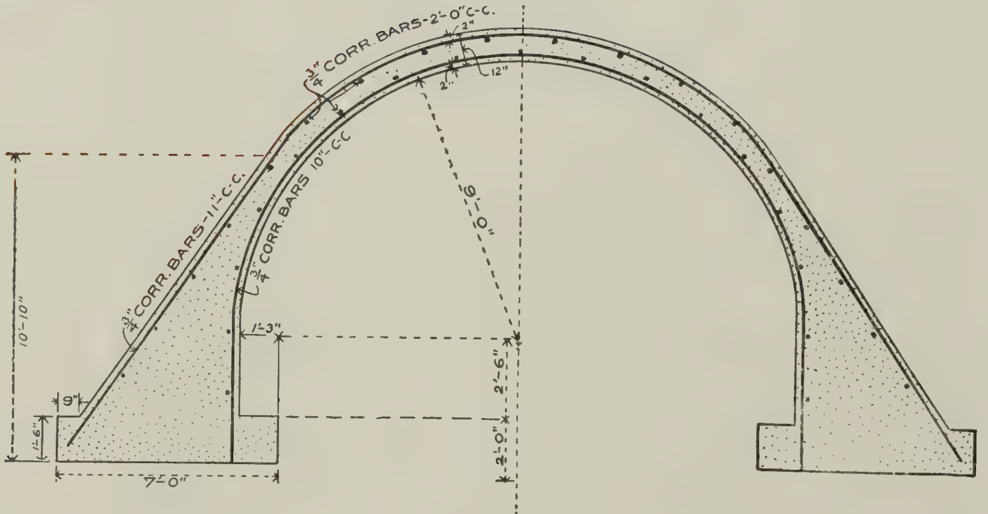
Section of Highway Culvert Construction. Marion Co., Ind.
H. W. Klausmann, County Engr.

Completed Culvert, Marion, Co., Ind.



Cross Section of Highway Bridge Floor Construction. Designed for Cooper's Class A specifications. Many floors like this have been built.

Expanded Metal Floor Construction on Highway Bridge at Waco, Texas. Span 535 Feet.

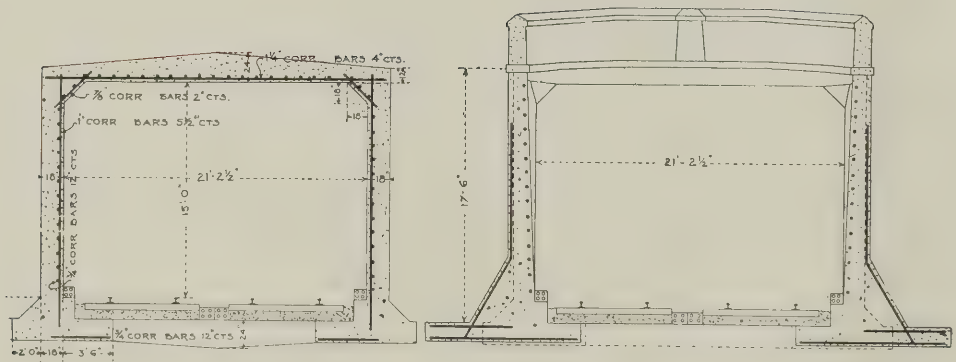


Section of Highway Culvert at South Bend, Ind. A. J. Hammond, City Engr.

Completed Culvert, South Bend, Ind.

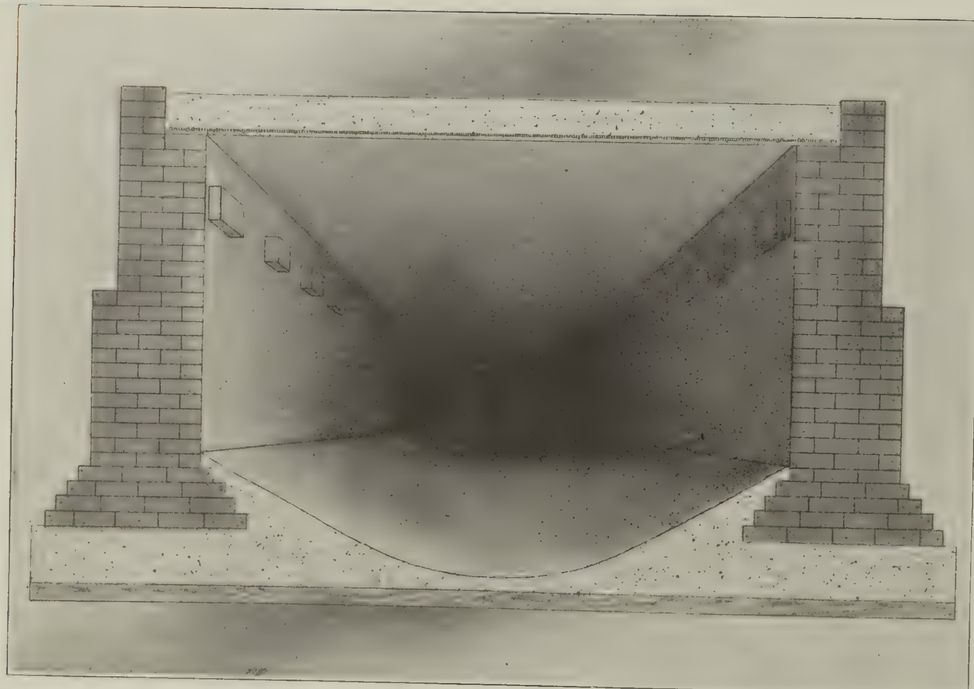
Highway Bridge, Anderson County, Kansas, Kansas City Bridge Co.

Boston Rapid Transit Subway. Howard A. Carson, Chief Engineer.



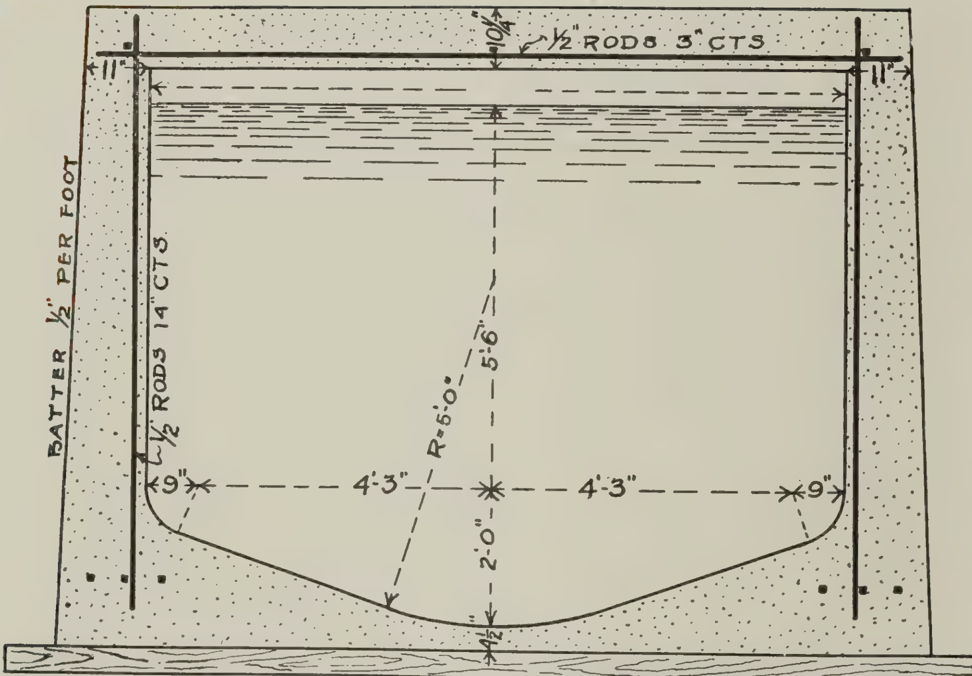
Section of Tunnel and Retaining Wall, Metropolitan Street Railway Co., Kansas City, Mo.
Ford, Bacon & Davis, Engineers.

Metropolitan Street Railway Company Tunnel.



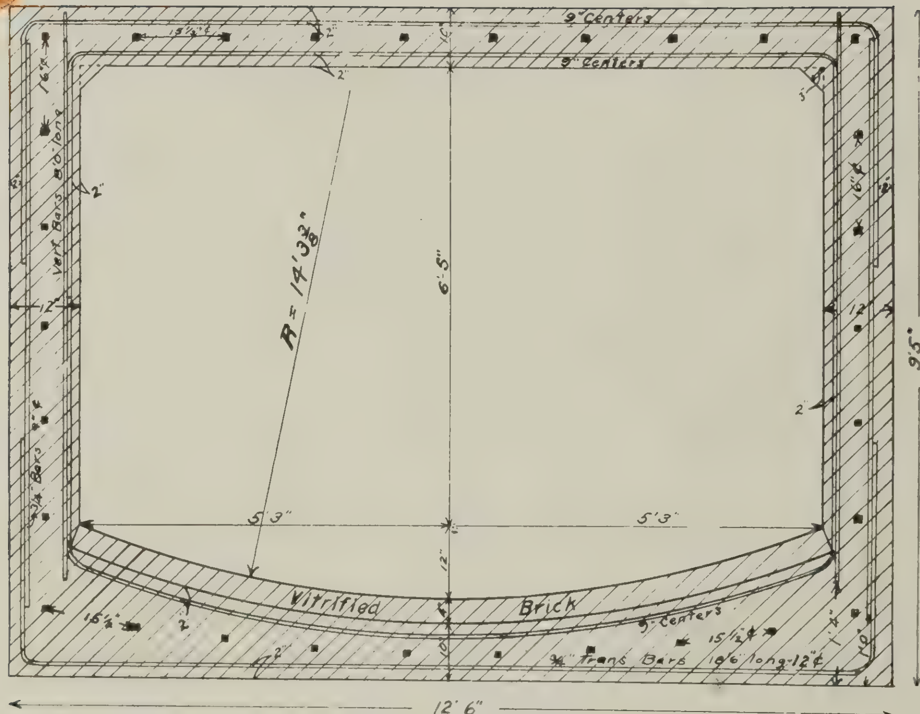
Section of New Orleans Drainage Canal. Maj. B. M. Harrod, Chief Engineer.

New Orleans Drainage Canal, Showing Test. Gravel Concrete, 1:3:6; span, 13′; slab, 11¼″ thick; reinforcement, ½″ □ corrugated bars, 4¾″ cts.; load, 51,150 pounds on two 8″x8″ supports in center, 6 feet apart. Deflection scarcely appreciable.



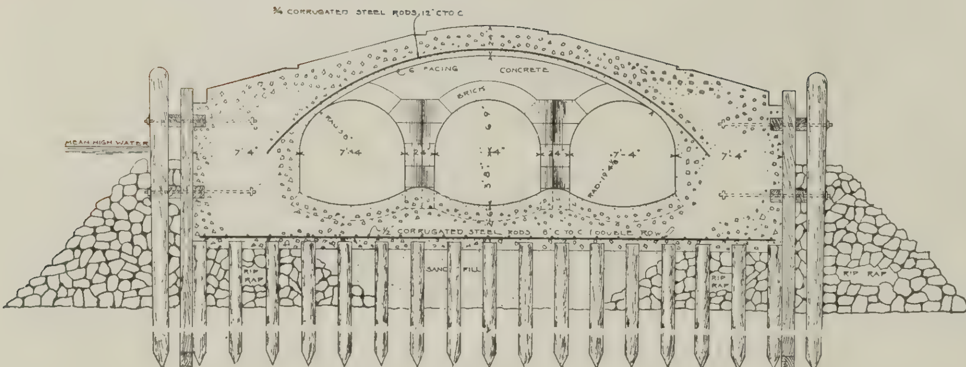
Last Type of New Orleans Drainage Canal.

Last Type of New Orleans Drainage Canal under Construction.



St. Louis Terminal Railway Association—Section of Sewer under Baggage Floor.
J. L. Armstrong, Engr. M. of W. A. P. Greensfelder, Asst. Engr.

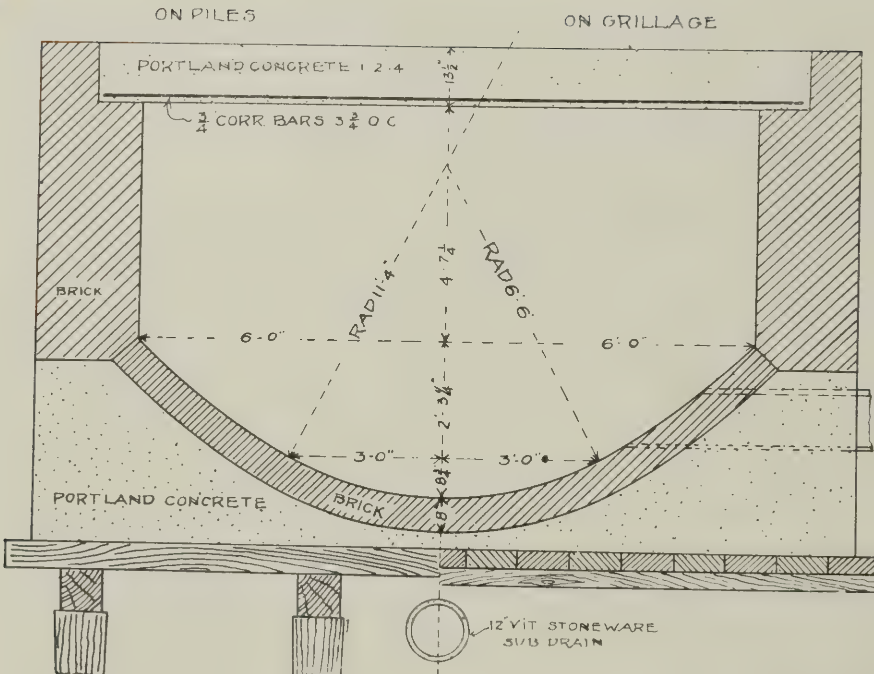
St. Louis Terminal Railway Association—Meeting Point of Two Branches of Sewer.



SECTION MAIN OUTLET SEWER BROOKLYN NEW YORK.

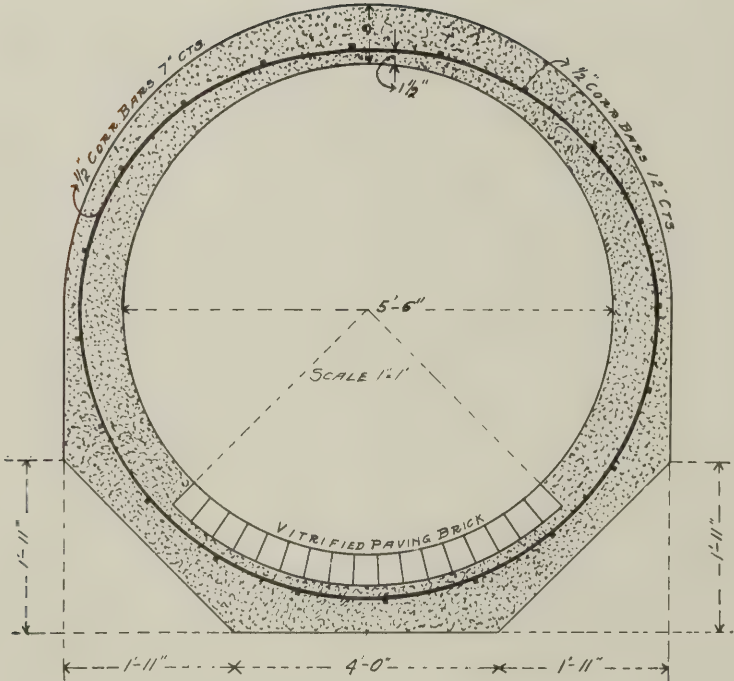
R. H. Asserson, Chf. Engr.

Main Outlet Sewer, Brooklyn, during Construction.

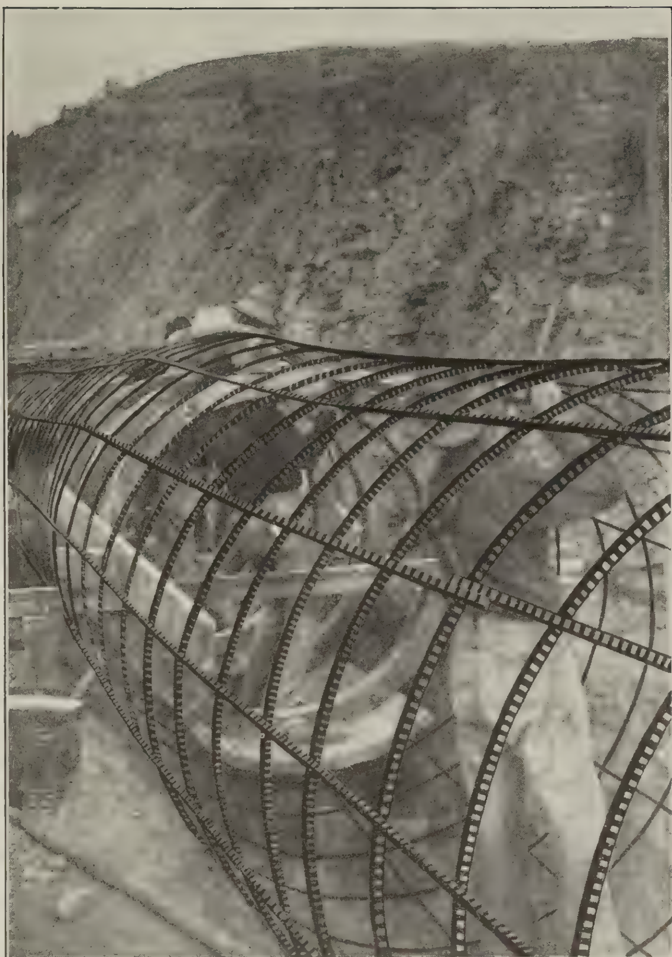


R. H. Asserson, Chf. Engr.

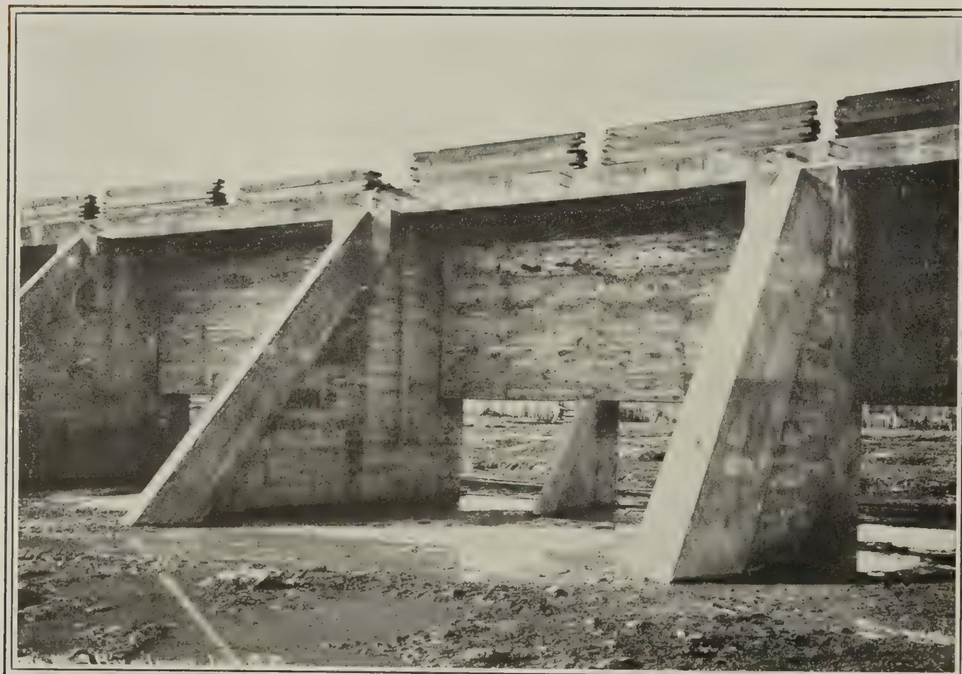
Another Type of Brooklyn Sewer Construction.



Cross Section of Conduit at Del Rio, Texas. J. W. Maxcy, Engineer.

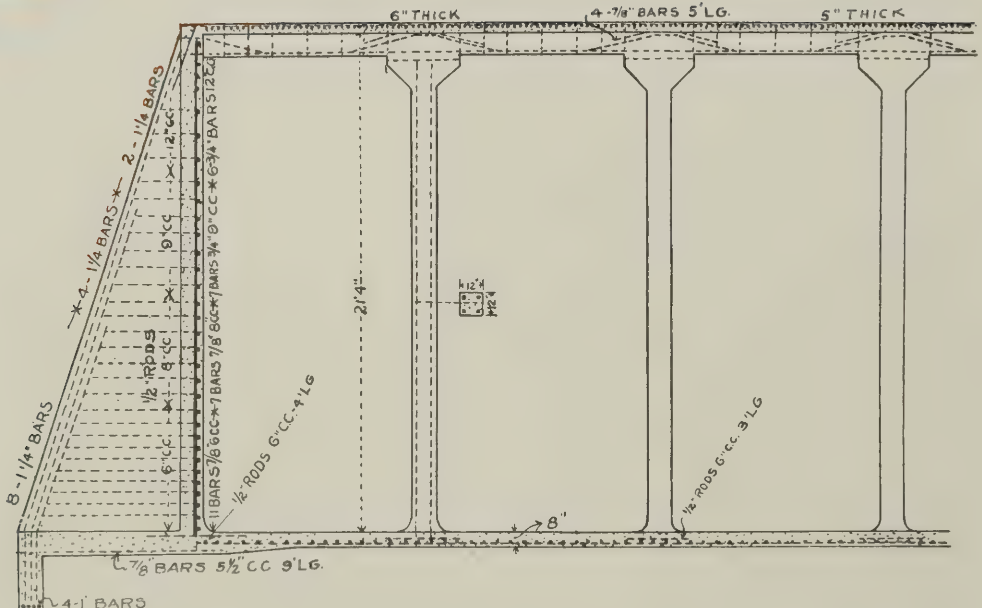


Del Rio Conduit under Construction.

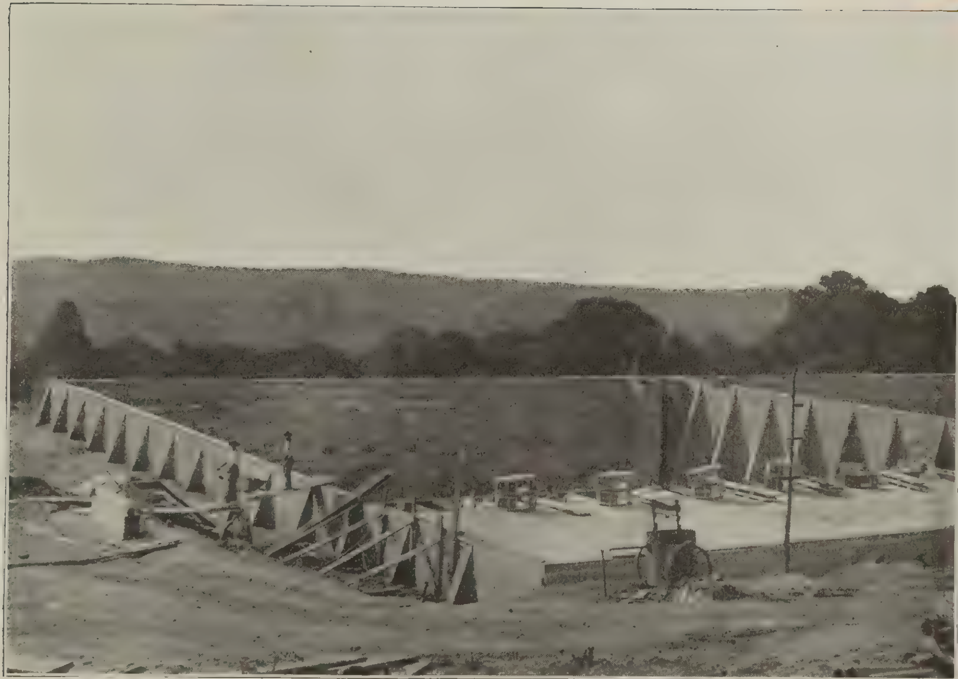


Detail of Intake, Ontario Power Co., Niagara Falls, Can.

Intake, Ontario Power Co., Niagara Falls, Canada.
L. L. Nunn,
P. N. Nunn, Engineers.



Section of Reservoir Construction, East Orange, N. J.
C. C. Vermeule, Conslt. Engr.
Commonwealth Roofing Co., Contrs.

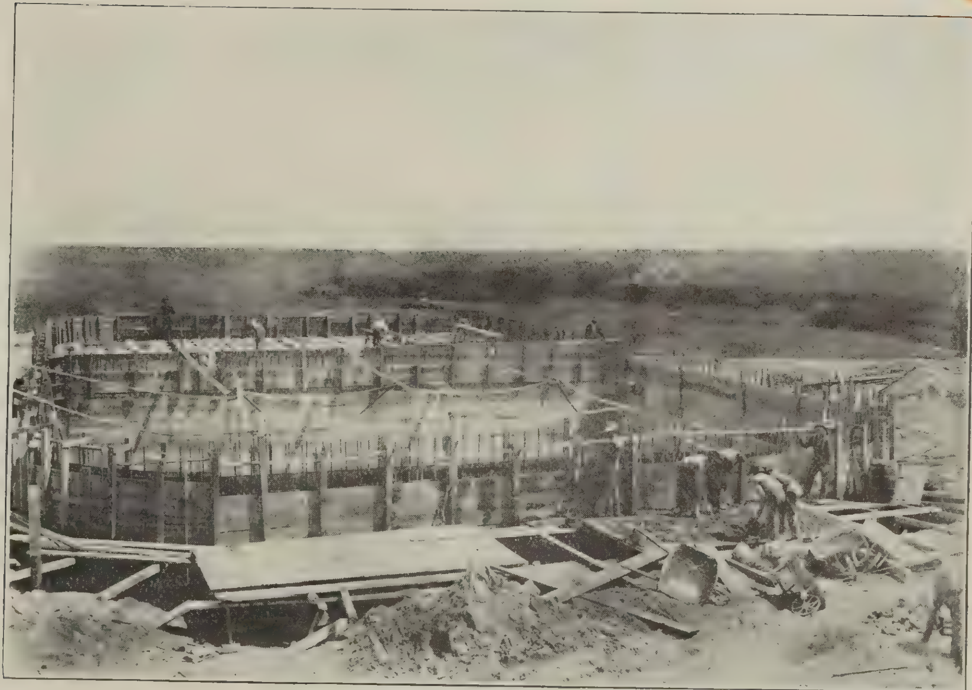


East Orange Reservoir under Construction.

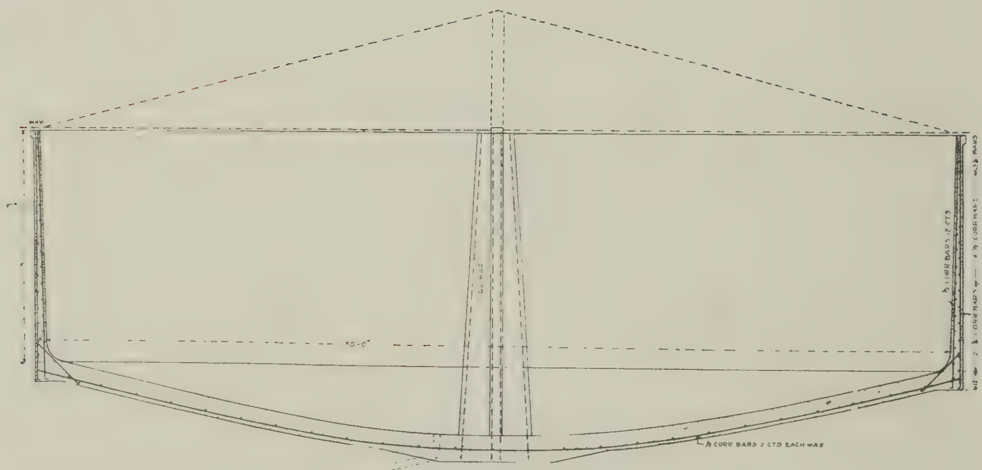
Top View of Reservoir Roof Under Construction, Indianapolis Water Co.

End View Dividing Wall, 350 feet long, Indianapolis Water Co., Reservoir.
Designed by T. L. Condron.
Built by Ind. Water Co.

Interior View, Reservoir, Indianapolis Water Co.

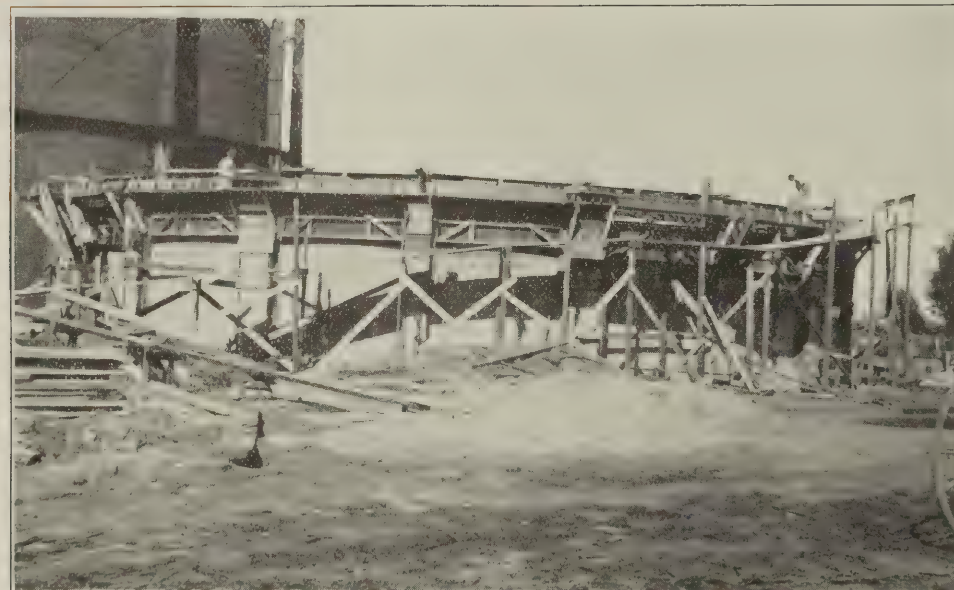


Ft. Meade Reservoir, Under Construction.
Designed by St. Louis Ex. M. F. P. Co.
Built by Dunnegan and Sykes.

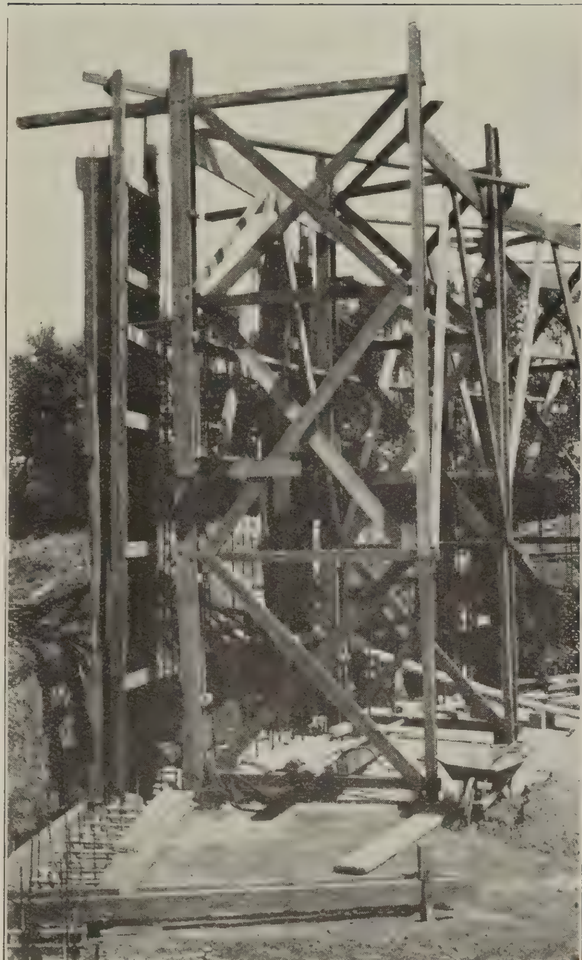


Reservoir at Lake Geneva, Wis. A. C. Warren, Engr.

Photograph of Completed Lake Geneva Reservoir.



Gasholder Tank, Key City Gas Co., Dubuque.
Geo. McLean, Pres. and Gen. Mgr.
Designed by St. Louis Ex. M. F. P. Co.
Built by Key City Gas Co.
J. E. Conzelman, Engr. in Charge Constr.



Gasholder Tank, Key City Gas Co.
Interior view, showing false work and method of erection.

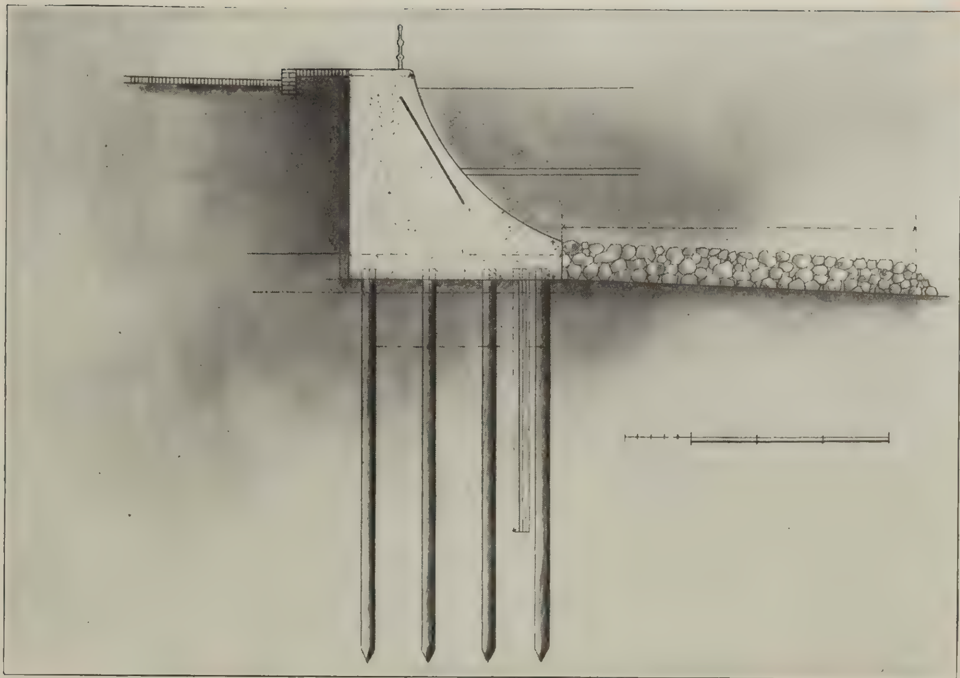
Gasholder Tank, Key City Gas Co.
Wall and Pilaster Forms.

Bordentown Water Tower.
C. C. Vermeule, Engr.
H. J. Riley, Jr., Contractor.

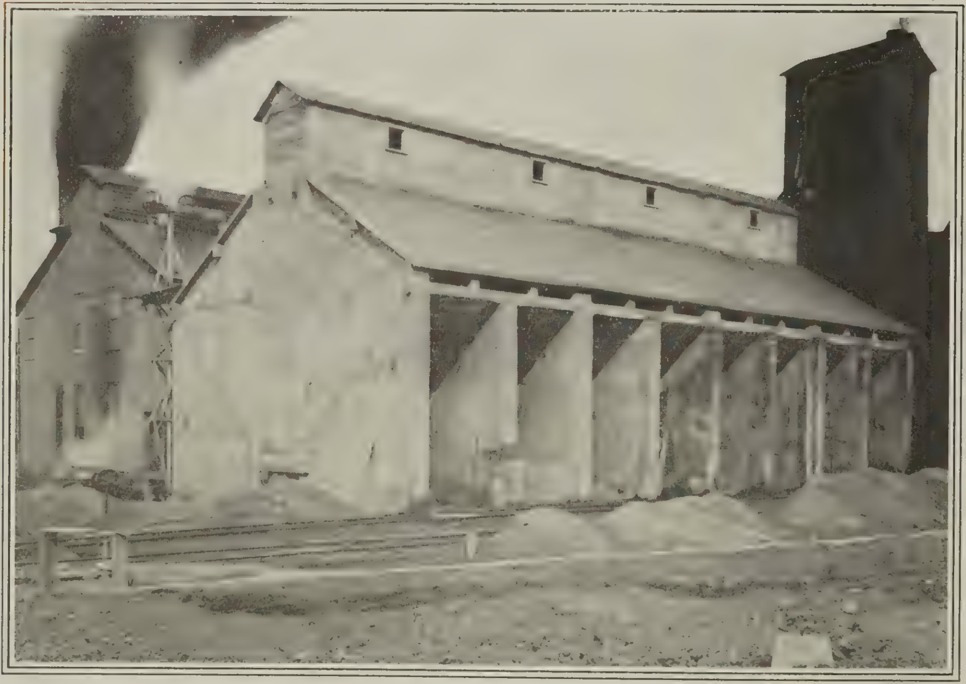
Photograph of Completed Tower.
Water Works, East Orange, N. J.
C. C. Vermeule, Conslt. Engr. Commonwealth Roofing Co., Contrs.

125′ Stack Under Construction for St. Louis Brewing Association.
Gilsonite Constr. Co., Contrs.

Galveston Sea Wall during Construction.

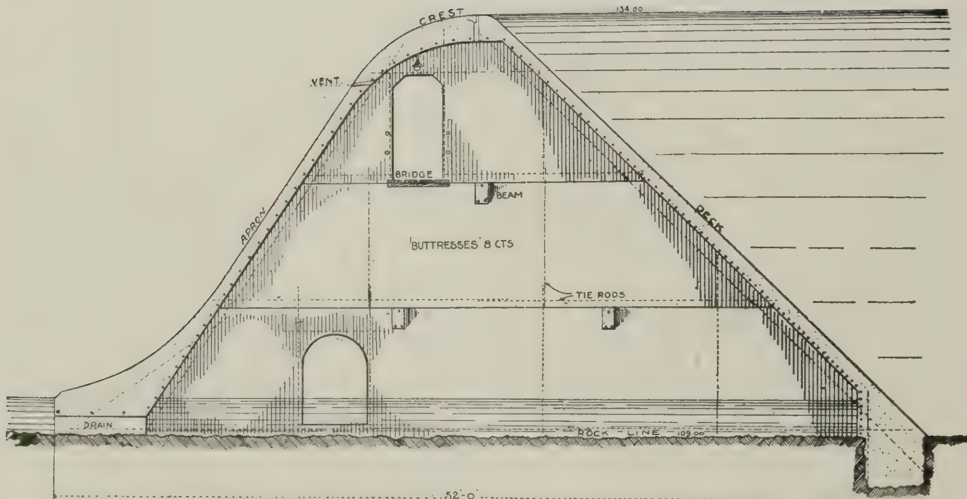


Galveston Sea Wall. Geo. W. Boschke, Engr. of Constr.

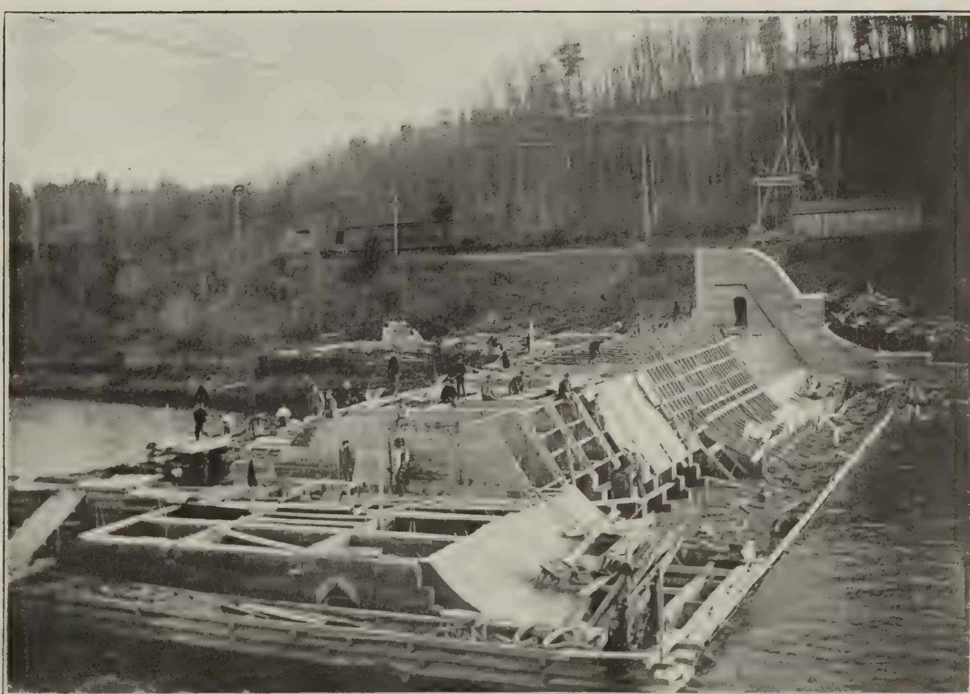


Coal Pockets for Pennsylvania Cement Co.
H. C. Miller, Engineer.
F. A. Little, Supt. of Construction.

Missouri Pacific Grain Bins at Kansas City.
Metcalf and Metcalf, Engineers.



Reinforced Concrete Dam Across the Battenkill,
Built for the American Wood Board Co., Schuylerville, N. Y.
Patented by Ambursen Hydraulic Construction Co., Boston, Mass.



Ambursen Dam at Schuylerville under Construction.

Palmer Lake Dam, Pueblo Div. D. & R. G. Ry. Span 110 feet, height 43 feet above outlet.
E. J. Yard, Chief Engineer.
W. A. Morey, Eng. B. and B.

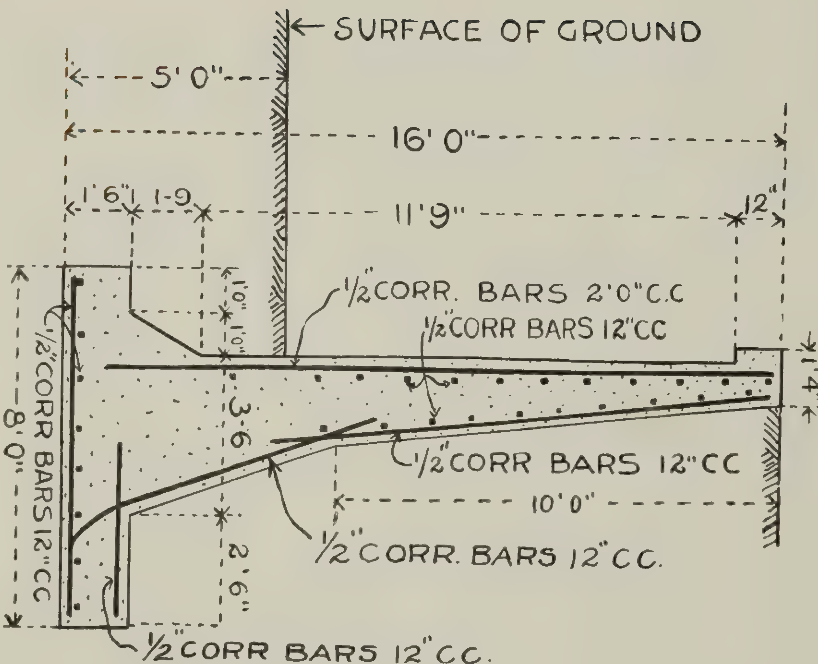
CONTINUOUS WALLS

One of the great advantages of reinforced concrete is in our ability to dispense with expansion joints in long structures. These may be built with the material in one piece from end to end, *a mile long if desired*, and by a properly proportioned longitudinal metal reinforcement, *shrinkage and temperature cracks can be entirely obviated.*

Most engineers have to be shown; and they will not believe it then unless they can see some scientific explanation of the matter. That explanation is as follows:

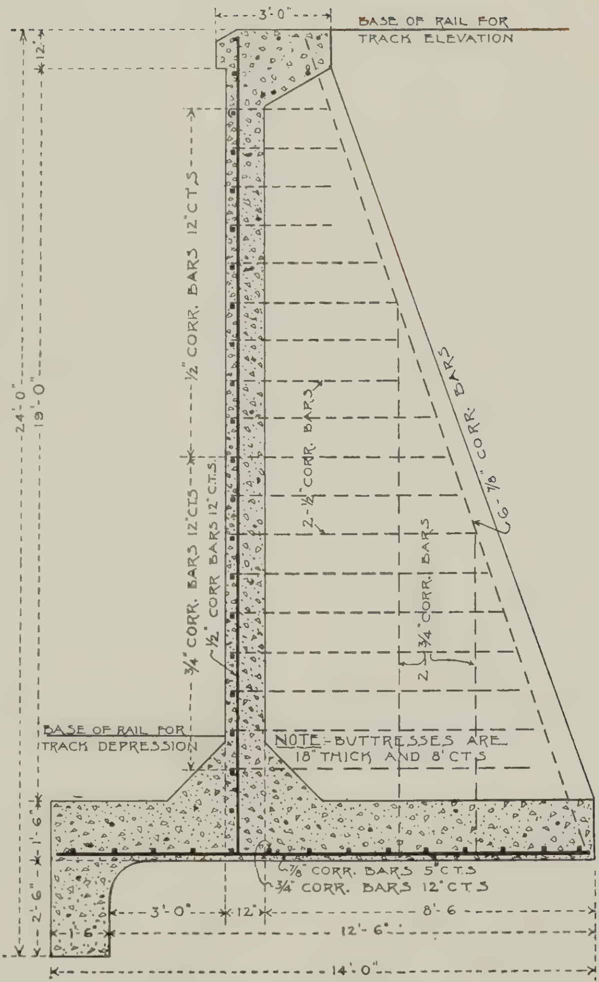
It has been shown by Considère, Hatt, and others, that concrete, when reinforced with metal well disseminated in small areas, will apparently stretch about ten times as much as when no metal is present, and that it will submit to proportionate elongations of about .0015. The co-efficient of expansion of concrete being .0000055, we find that *it would take a fall of 270° to develop a proportionate shortening equal to the wall's ability to stretch.* The wall will pull out in this manner at about three-fourths its full tensile strength, or say at 150 pounds per square inch.

The quantity of metal needed is enough to equal the tensile strength of the wall at an elongation of .0015, corresponding to a stress per square inch in the metal of 45,000 pounds. The area of metal would therefore be $\frac{1}{300}$ part of the area of the wall.



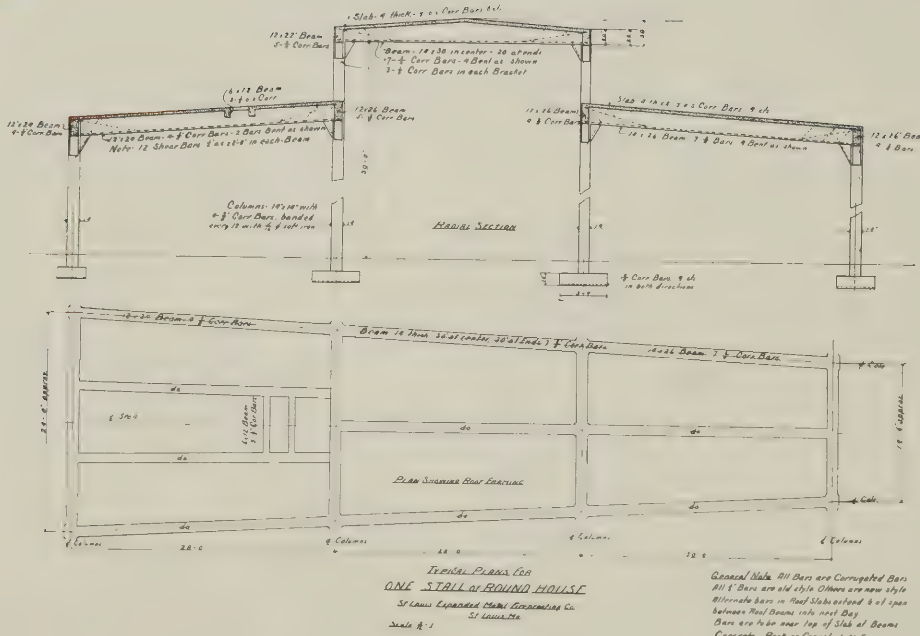
Section of Retaining Wall, Marion County, Ind.
H. W. Klausmann, County Surveyor.

Retaining Wall, Marion County, Ind.

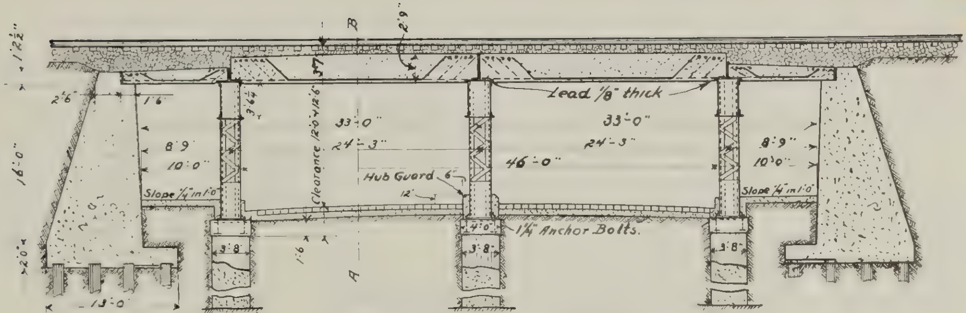


Retaining Wall for Track Elevation or Depression.
No expansion joints needed.

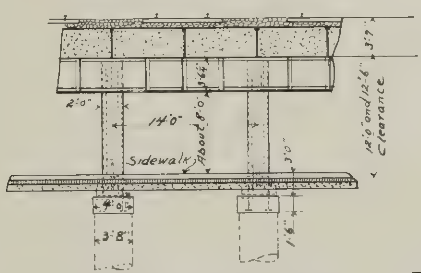
RAILROAD STRUCTURES



Typical Round House Details.

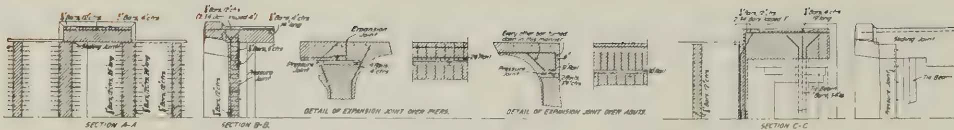


CROSS SECTION OF SUBWAY

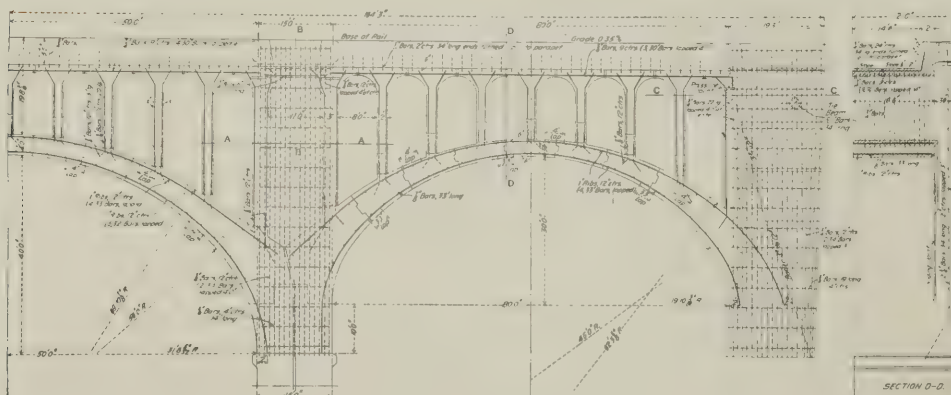


SECTION ON LINE A-B.

C. B & Q RY.
TRACK ELEVATION
CANAL ST. TO WESTERN AV.
TYPICAL SECTIONS AT SUBWAY.



Details of Steel Reinforcement in Vermillion River Concrete Arch Bridge



Details of Steel Reinforcement in Vermillion River Concrete Arch Bridge

Big Four Double Track R. R. Bridge near Danville, Ill.

Big Four Double Track R. R. Bridge., near Danville, Ill. Two 80-foot Spans, One 100-foot Span.
W. M. Duane, Engineer of Construction.
Bates and Rogers, Contractors.

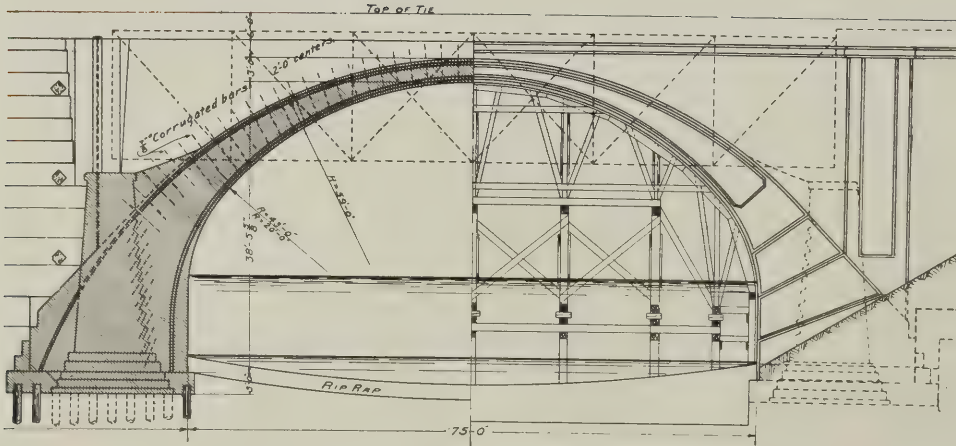
Four-Track Reinforced Concrete Arch at Willoughby Run on L. S. & M. S. R. R.
Clear Span, 154′.

E. A. Handy, Chief Engineer.
Frank Beckwith, Engr. of Bridges.

Willoughby Run Arch Completed.

Angola Reinforced Concrete Arch, L. S. & M. S. R. R.
E. A. Handy, Chief Engineer.
Frank Beckwith, Engineer of B. & S.

Approach to Bridge Across Mississippi River at Thebes, Ill.

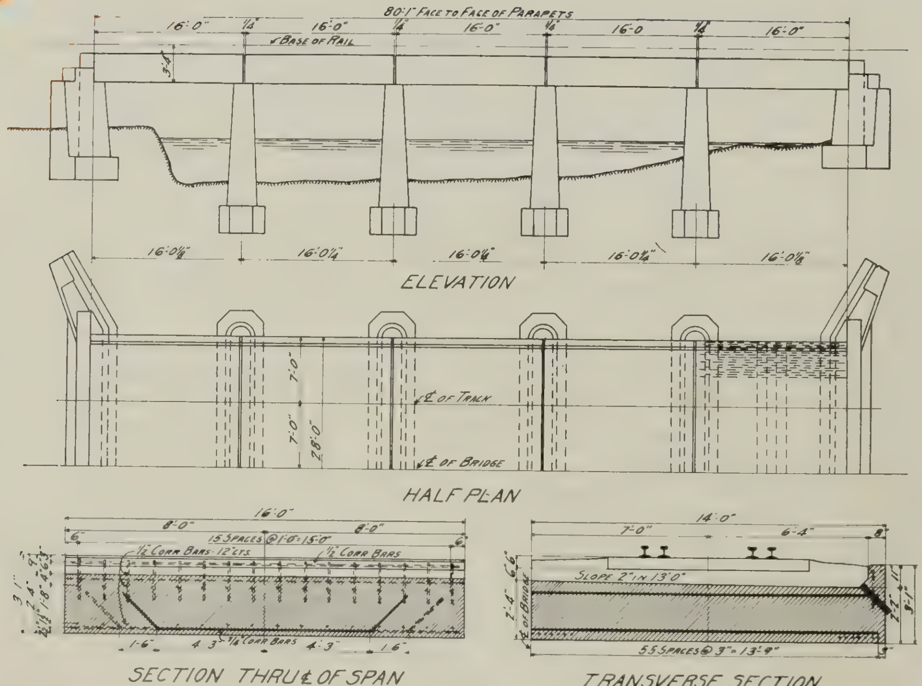


Plano Arch, 75′ Span, C., B. & Q. R. R. W. L. Breckenridge, Chief Engineer.
C. H. Cartlidge, Bridge Engineer.

Completed Plano Arch.

Reinforced Concrete Arch, C. & E. I. R. R., 56-foot Span.
W. S. Dawley, Chief Engineer.
Hoeffer & Co., Contractors.

Reinforced Concrete Arch, 75′ Span, on the Illinois Central Railway. H. U. Wallace Chf. Engr.
H. W. Parkhurst, Bridge Engr.

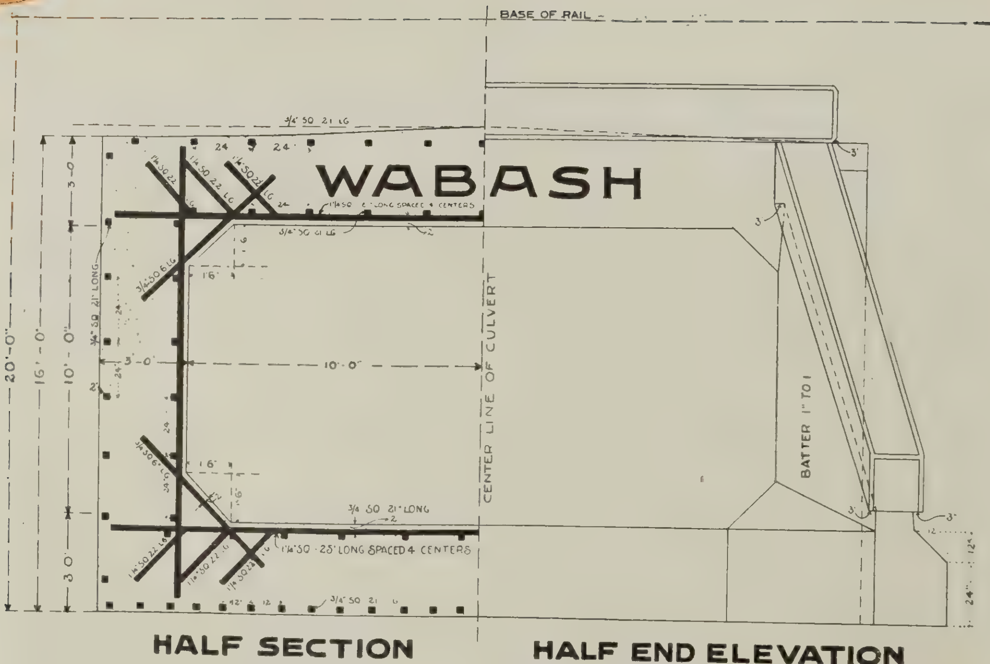


Reinforced Concrete Trestle, C., B. & Q. Ry., over Cave Hollow.
W. L. Breckenridge, Chief Engineer.
C. H. Cartlidge, Bridge Engineer.

Reinforced Concrete Trestle, C., B. & Q. Ry., over Cave Hollow.
W. L. Breckenridge, Chief Engineer.
C. H. Cartlidge, Bridge Engineer.

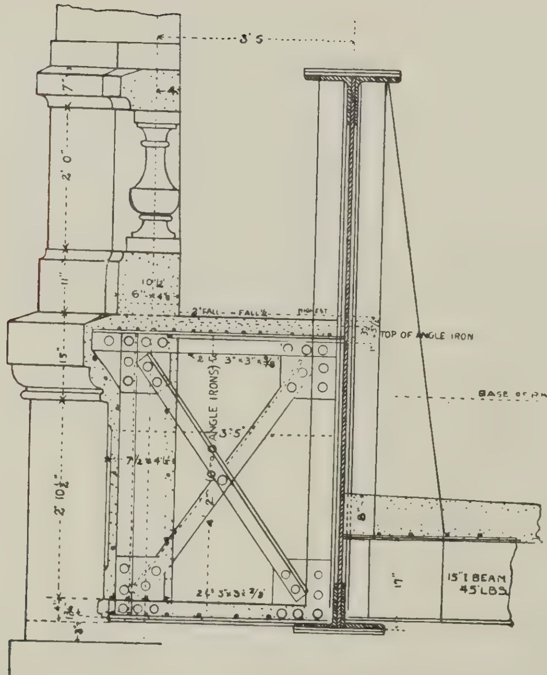
Subway Under C., B. & Q. Tracks, at Galesburg, Ill.
W. L. Breckenridge, Chief Engineer.
C. H. Cartlidge, Bridge Engineer.

Overhead Crossings, Big Four Ry., Short Line Between Lomax and Hillsboro.
W. M. Duane, Supt. of Construction.



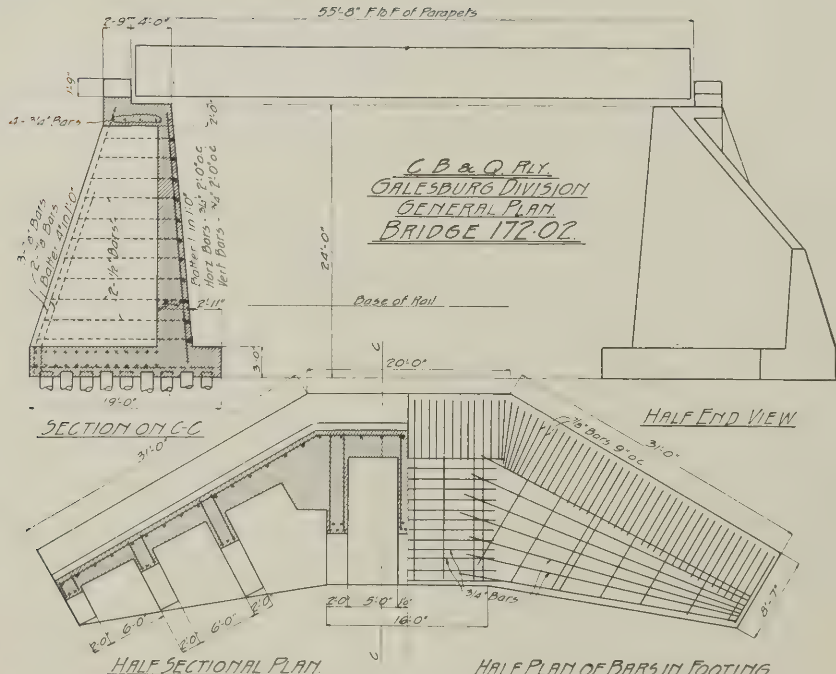
Section of Flat Top Culvert, 20′ Span, Wabash R. R., near St. Louis, Mo.
W. S. Newhall, Chief Engineer.
A. O. Cunningham, Bridge Engr.

Completed 20′ Culvert, Wabash R. R.



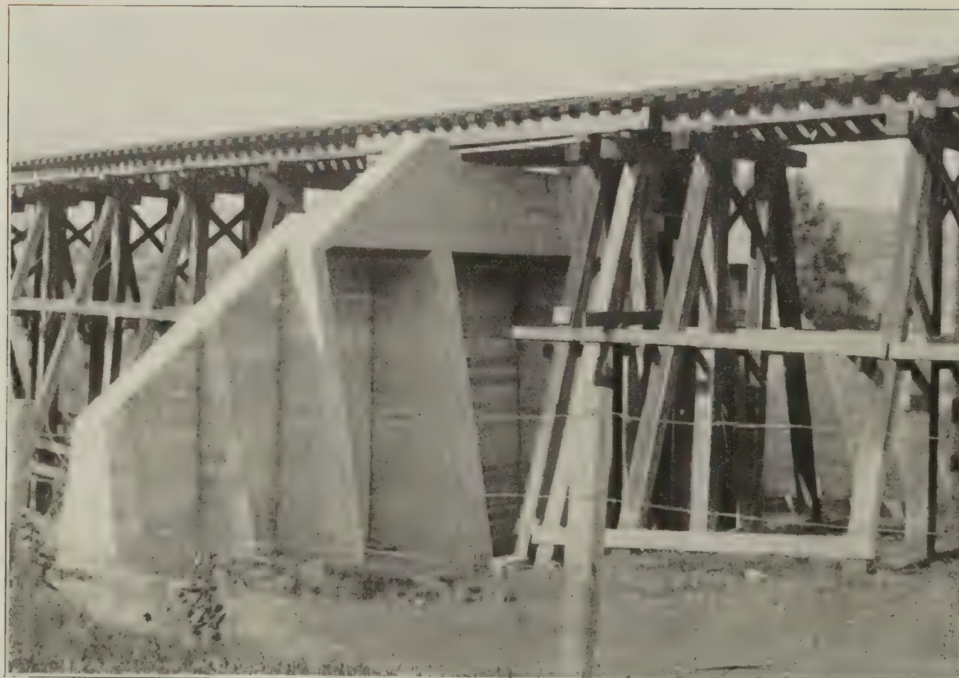
Wabash Plate Girder Bridge with Reinforced Concrete Floor, Hollow Abutments and Ornamental Balustrade, in Forest Park, St. Louis.
W. S. Newhall, Chief Engineer.
A. O. Cunningham, Bridge Engr.

Completed Wabash Bridge, Forest Park, St. Louis.



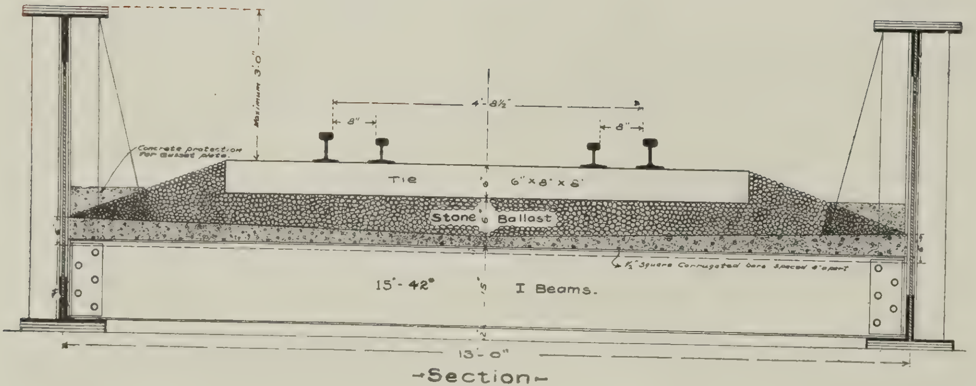
Reinforced Concrete Abutment, C., B. & Q. Ry.
W. L. Breckenridge, Chief Engineer.
C. H. Cartlidge, Bridge Engineer.

40-Foot Abutments, Illinois Terminal Railway.
T. C. Moorshead, Chief Engineer.
Myers Construction Co., Contractors.

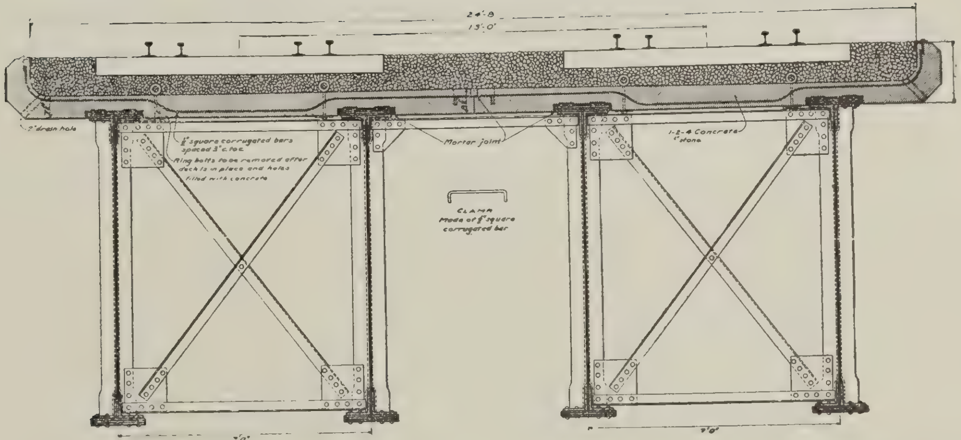


Abutment, N. O. & W. R. R.
C. E. Knickerbocker, Engr. M. of W.

Three Track R. R. Arches, C. & E. I. R. R. 20′ 6″ Spans.
W. S. Dawley, Chief Engineer.
Designed by T. L. Condron.
Built by Railroad Co.



One Type of Solid Reinforced Concrete Bridge Floor, Wabash Railroad.
W. S. Newhall, Chief Engineer.
A. O. Cunningham, Bridge Engr.



One Type of Solid Reinforced Concrete Bridge Floor on the C. B. & Q. R. R.
W. L. Breckenridge, Chief Engineer; C. H. Cartlidge, Bridge Engineer.

REINFORCED CONCRETE BEAMS

The number of variables entering into the discussion of the resisting moment of reinforced concrete beams makes it impracticable to develop a general formula that will correctly give the stress values at all stages of loading. However, by assuming a definite law of variation between stress and the corresponding deformation of the concrete, the resisting moment can be evaluated for any given percentage of reinforcement by further assuming the stress in the steel. The principle of invariability of plane sections, together with the statical requirement, that the total tension must be equal to the total compression, fixes the position of the neutral axis. The resisting moment is then determined by taking moments either about the neutral axis or about the centroids of compression and tension.

A great variety of assumptions have been made regarding the relation between stress and strain. The tendency at present is to consider this relation as represented either by a straight line or a parabola, and also to neglect the value of concrete in tension. In any case, the area of the stress strain curve must be found, and the position of its center of gravity located. It is evidently inconsistent to arbitrarily assume these values without any regard to the form of the compression area. It is equally inconsistent to assume a rectilinear stress strain diagram and then express the value of the total compression by anything but $\frac{1}{2} f_c b y_1$. After due consideration

of experimental data regarding the form of the stress strain curve and the actual carrying capacity of reinforced beams, the most reasonable assumptions appear to be: That the compressive stresses vary as the ordinates to a parabola whose vertex is either at the top of the beam or above; and that the concrete is subjected to tensile stress from the neutral axis to a point in the section where the elongation is the same as that developed by a plain beam in cross bending.

Most formulæ for the strength of reinforced concrete beams are based upon a rectilinear relation between stress and strain, and the *safe* values inserted therein, instead of the *ultimate* values. In our judgment this is not wise, as it is impossible to know what factor of safety is obtained. Most of these formulæ will take 16,000 pounds per square inch for the safe stress in the steel and say that there will be a factor of safety of four on the structure, because the ultimate strength of the steel is 64,000 pounds per square inch. But when the elastic limit of the metal is passed *its modulus drops from* 30,000,000 *to* 5,000,000 and the cracks in the concrete become so very large immediately that we do not consider as *available* any strength that can be obtained beyond this limit; though this excess is considerable if the quantity of reinforcement used is only *one-half* what it should be, as is the case in the method above described. With only one-third the quantity of metal necessary to develop the required ultimate strength at the elastic limit, it is possible to break the metal entirely in two. For example, in a

six-inch slab of rock-concrete having expanded metal embedded in its lower portion, the expanded metal will always be broken apart, though this is soft box-annealed material. But the factor of safety for such construction should be four on the elastic limit, which would be equivalent to about six on the maximum load. When, therefore, we give the beam credit for no more strength than it can develop at the elastic limit of the steel reinforcement, it is desirable that this limit should be fairly high. With an elastic limit of over 30,000 pounds per square inch the most economical quantity of metal reinforcement is 1.4 per cent of the area of the concrete, while with a limit of 50,000—0.7 per cent only is required, or a saving of approximately one-half in the cost of the metal.

As has been stated in the introduction, there is still some discussion as to just when the first crack develops in reinforced concrete: but as also there shown, a proper reinforcement will cause the beam to develop a large number of cracks very close together, in which case these cracks will be of no material consequence so long as the bars are stressed inside the elastic limit. Corrugated bars will accomplish this result. The cracks will be close together, small in size, and will not be able to reach the bar itself. With plain bars, or bars of less positive form of bond, this is not true; and beams reinforced with such material cannot demonstrate immunity from injury even if the stress in the bars is inside the elastic limit. Such beams exposed to the

action of the atmosphere would be liable to have the reinforcement much corroded in time.

In the following discussion it is assumed that a section plane before bending is plane after bending. It is further assumed that the modulus of elasticity of concrete varies, its value decreasing as the stress increases, and that its instantaneous value may be represented by the tangent to a parabola.

To obtain an equation for a parabola that would represent the variations of the modulus, an inspection of a number of stress-strain diagrams was made, which led to the conclusion that if the modulus at rupture was taken as two-thirds of the initial modulus, the parabola so obtained would represent closely the actual stress-strain diagram. The tensile stresses in the concrete, between the neutral axis and that plane at which the unit elongation has the limiting value λt, are considered in the discussion.

We have, then, for Rectangular Beams, the following discussion:

RECTANGULAR BEAMS.

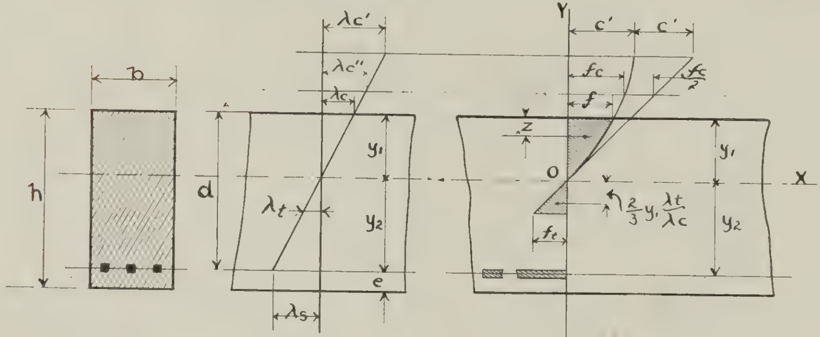


Fig. 1 Fig. 2 Fig. 3.

Fig. 1 is a cross section of a reinforced concrete beam.

Fig. 2 represents the strain or deformation diagram at any instantaneous load.

Fig. 3 is the stress diagram corresponding to the above strain diagram.

Let E_s=Modulus of elasticity of steel in pounds per square inch.

E_c=*Initial* modulus of elasticity of concrete in compression in pounds per square inch.

F=Elastic limit of steel in pounds per square inch.

f_c=Compressive strength of concrete in pounds per square inch.

f=Compressive stress on extreme fiber in pounds per square inch. f may have any value less than f_c.

c'=Abscissa to stress diagram at vertex of parabola.

s=Any assumed unit stress in steel, pounds per square inch.

f_t=Modulus of rupture of concrete in cross bending, in pounds per square inch.

λ_c=Unit deformation of extreme compression fiber corresponding to a stress f.

λ_c''=Unit deformation of extreme compression fiber at ultimate stress f_c.

λ_c'=Unit deformation corresponding to stress ${}_c'$. Note that λ_c' and ${}_c'$ deal with conditions after the ultimate strength of the concrete is passed, and have no value except in determining the curve, etc.

λ_t=Unit elongation of concrete corresponding to stress f_t.

λ_s=Unit elongation of steel corresponding to stress s.

b=Width of beam in inches.

z=Distance from top fiber to center of gravity of compression area in inches.

y_1=Distance from neutral axis to extreme fiber in compression in inches.

y_2=Distance from neutral axis to plane of reinforcement in inches.

e=Distance in inches from plane of metal to extreme fiber on tension side.

$d=y_1 + y_2$ = Effective depth of beam.

p=Ratio of reinforcement in terms of $bd = q \div bd$.

a=Ratio of reinforcement in terms of $bh = q \div bh$.

M=Bending moment of external force in inch pounds=resisting moment of beam.

M_o=Ultimate moment of resistance of cross section in inch pounds.

P_s=Total stress in metal in width b.

P_c=Total compressive stress in concrete in width b.

P_t=Total tensile stress in concrete in width b.

q=Area of metal in width b, in square inches.

Referring to Fig. 3, the shaded area above the neutral axis represents the compressive stress diagram of the concrete, $o\ y$ being the

axis of proportionate elongation, and $o\ x$ the axis of stress per square inch.

Before getting the area of the compression diagram, it will be necessary to get the equation of the parabola referred to the axis $o\ x$ and $o\ y$. We have E_c, which is represented by the tangent to the parabola at the origin 0, and have also imposed the condition that the final modulus at rupture is two-thirds the initial modulus. The equation for the parabola then becomes:

$$f = E_c \lambda_c - \frac{E_c \lambda_c^2}{2 \lambda_c'}$$

$$f_c = \tfrac{2}{3} E_c \lambda_c''$$

From which $\lambda_c'' = \dfrac{3 f_c}{2 E_c}$ .... (1)

And $\lambda_c' = \dfrac{9 f_c}{4 E_c}$ .... (2)

Substituting in the general equation and solving for λ_c we get

$$\lambda_c = \lambda_c' \left(1 - \sqrt{1 - \frac{8f}{9f_c}}\right) = \lambda_c' r \quad \text{.......... (3)}$$

We can now get a value for y_1 : —

From the strain diagram,

$$\frac{y_1}{d - y_1} = \frac{\lambda_c}{\lambda_s}$$

Or $$y_1 = \frac{\lambda_c d}{\lambda_s + \lambda_c}\text{, but } \lambda_s = \frac{s}{E_s}$$

Therefore, $$y_1 = \frac{E_s \lambda_c}{s + E_s \lambda_c} d \quad \text{...(4)}$$

The expression for the area of the compressive stresses may be written in the form,

$$\frac{P_c}{b} = \left(1 - \frac{\lambda_c}{3\lambda_c'}\right)\frac{E_c \lambda_c}{2} y_1 = Dy_1 \quad \text{.................................(5)}$$

For the area of the tensile stresses we may, without appreciable error, consider the parabolic area as a triangle (since the allowed stress is very small, the tangent and parabola practically coincide), and can express the area by the equation,

$$\frac{P_t}{b} = \frac{f_t}{2} \frac{\lambda_t}{\lambda_c} y_1 = \frac{E_c \lambda_t^2}{2 \lambda_c} y_1 = Gy_1 \quad \text{...(6)}$$

Since the sum of the compressive and tensile stresses must equal zero, we can write $P_s = P_c - P_t$

therefore $p = \frac{y_1}{d} \times \frac{D\text{-}G}{s}$ ....(7)

We have, taking moments about the center of gravity of the compressive stresses, the following expression for the moment of resistance of the section,

$$M = P_s\,(d - z) + P_t \left(\frac{2\,\lambda_t}{3\,\lambda_c}\, y_1 + y_1 - z \right)$$

$$= pbds\,(d - z) + by_1\,G\,\left(y_1 + \frac{2}{3}\, y_1\, \frac{\lambda_t}{\lambda_c} - z\right) \text{ ....(8)}$$

$$z = \frac{4 - r}{12 - 4r}\, y_1, \text{ where } r = \frac{\lambda_c}{\lambda_c'} \text{ ....(9)}$$

It is to be noted that the above discussion is perfectly general, and we may, by assuming any fiber stress f, and any stress in the steel, s, find the percentage of reinforcement required, and the resisting moment of the section.

We are, however, mainly interested in the ultimate strength of the beam, reinforced with the critical percentage of metal (it being

taken for granted that the designer will apply his factor of safety to the actual moments, designing the section for the ultimate moment so obtained), which condition obtains when the percentage of steel is so chosen that the beam is equally strong in tension and compression, or differently expressed, that the stress in the steel reaches the elastic limit at the same time that the compressive stress on the extreme fiber becomes the ultimate strength of the concrete.

Putting these values in the general equation, No. 8, we get the following:

$$M_o = pdbF(d-z) + by_1G(y_1 + \tfrac{2}{3}y_1 \frac{\lambda_t}{\lambda_c} - z) \qquad \text{(8a)}$$

The size of beam needed to develop a required moment of resistance can be obtained from the above equations, when the constants dependent upon the particular materials used are known.

AVERAGE ROCK CONCRETE.

We have taken as the best average values for the constants for 1 : 3 : 6 concrete the following: $E_c=2{,}600{,}000$, $f_c=2000$, and $\lambda_t=.00015$. For the steel the value of E_s is practically constant for all grades of

material, but F, or the elastic limit, varies greatly. Since we cannot utilize any of the strength of the steel beyond the elastic limit, it is desirable to have this limit fairly high. Our corrugated bars have an elastic limit of between 55,000 and 65,000 pounds per square inch. We therefore use for the constants for steel, $E_s=29{,}000{,}000$ and $F=55{,}000$.

With these values we can derive the following equations:

$$\lambda_c'=0.0017308, \qquad \lambda_c''=0.0011539$$

$$\left.\begin{array}{l} y_1=.3782d \\ \\ q=.00785bd \\ \\ M_o=376bd^2 \end{array}\right\} \begin{array}{l} \text{If } b=12'' \\ \\ \text{and } \rho=\dfrac{h}{10} \\ \\ \text{we have,} \end{array} \left\{\begin{array}{lr} y_1=.3404h \ldots\ldots & (10) \\ q=.08478h \ldots\ldots & (11) \\ \quad =.7065\% \text{ of} \ldots\ldots & (11a) \\ \quad \text{cross section} & \\ M_o=3655h^2 \ldots\ldots & (12) \\ h=0.01654\sqrt{M} \ldots\ldots & (13) \end{array}\right.$$

GOOD ROCK CONCRETE.

Using a 1 : 2 : 5 mix, and good rock or gravel, we get a concrete of much greater compressive strength, but with a higher modulus of elasticity. For such concrete we may assume the following constants:

$$E_c=2{,}800{,}000, \qquad f_c=2700, \qquad \lambda_t=.00015$$

Using the same values for the steel as before, our equations of design for ultimate load become,

$$\lambda_c'=0.002169, \qquad \lambda_c''=0.0014464$$

$$\left.\begin{array}{l} y_1=.433d \\ \\ q=.01223bd \\ \\ M_o=572bd^2 \end{array}\right\} \begin{array}{l}\text{If } b=12'' \\ \\ \text{and } e=\dfrac{h}{10} \\ \\ \text{we have,}\end{array} \left\{\begin{array}{ll} y_1=.3897h \ldots\ldots & (14)\\ q=.1321h \ldots\ldots & (15)\\ \quad=1.101\% \text{ of} \ldots\ldots & (15a)\\ \quad\text{cross section} & \\ M_o=5560h^2 \ldots\ldots & (16)\\ h=.01341\sqrt{M} \ldots\ldots & (17)\end{array}\right.$$

CINDER CONCRETE.

For a 1 : 2 : 5 mix of cinder concrete, we have $E_c=750,000$, $f_c=750$ pounds, and $f_t=75$. We find, however, that the final modulus for cinder concrete is one-half the initial, which modifies the previous equations slightly. Substituting these values in the equations, we get the following values for the ultimate moment of resistance of cinder concrete beams:

$$\lambda_c=\lambda_c''=0.0020$$

$y_1=.5133d$	If $b=12''$	$y_1=.462h$ .... (18)
		$q=.0503h$ .... (19)
$q=.00465bd$	and $e=\dfrac{h}{10}$	$=.42\%$ of cross section .... (19a)
		$M_o=2000h^2$ .... (20)
$M_o=207bd^2$	we have,	$h=.02236\sqrt{M}$ .... (21)

DESIGNING TABLES FOR AVERAGE AND GOOD ROCK CONCRETE.

The following table gives the necessary depth and the amount of reinforcement required for a beam 12 inches wide, corresponding to the ultimate resisting moments given.

TABLE FOR USE IN DESIGNING REINFORCED CONCRETE BEAMS. 12″ WIDE.

1 : 3 : 6 CONCRETE.						1 : 2 : 5 CONCRETE.					
M	*h*	*q*	*M*	*h*	*q*	*M*	*h*	*q*	*M*	*h*	*q*
50	3.70	0.314	1000	16.54	1.402	50	3.00	0.397	1000	13.41	1.772
100	5.23	.443	1500	20.26	1.716	100	4.24	.560	1500	16.42	2.170
150	6.41	.543	2000	23.40	1.985	150	5.20	.687	2000	18.96	2.506
200	7.40	.627	2500	26.16	2.218	200	6.00	.793	2500	21.20	2.803
250	8.27	.701	3000	28.66	2.430	250	6.71	.886	3000	23.23	3.070
300	9.06	.768	3500	30.90	2.620	300	7.35	.971	3500	25.10	3.318
350	9.79	.830	4000	33.10	2.806	350	7.94	1.048	4000	26 83	3.545
400	10.47	.887	4500	35.10	2.975	400	8.48	1.120	4500	28.45	3.760
450	11.10	.941	5000	37.00	3.135	450	9.00	1.188	5000	30.00	3.965
500	11.70	.992	5500	38.80	3.290	500	9.48	1.252	5500	31.45	4.155
550	12.26	1.039	6000	40.55	3.438	550	9.94	1.313	6000	32.85	4.340
600	12.81	1.086	6500	42.20	3.578	600	10.38	1.373	6500	34.20	4.520
650	13.34	1.131	7000	43.80	3.714	650	10.81	1.428	7000	35,45	4.685
700	13.84	1.173	7500	45.30	3.840	700	11.22	1.482	7500	36.70	4.850
750	14.33	1.215	8000	46.80	3.968	750	11.61	1.535	8000	37.90	5.010
800	14.80	1.255	8500	48.23	4.090	800	12.00	1.585	8500	39.10	5.165
850	15.25	1.293	9000	49.63	4.208	850	12.36	1.633	9000	40.25	5.320
900	15.70	1.331	9500	51.00	4.325	900	12.72	1.680	9500	41.35	5.465
950	16.12	1.367	10000	52.32	4.436	950	13.07	1.726	10000	42.40	5.605

The moments given in the table are the ultimate moments of resistance of the sections in thousands of inch pounds. To use table first apply desired factor of safety to actual moments.

M=Ultimate bending moment of external forces in thousands of inch pounds=Mo.

h=Depth of beam in inches: *d*=Depth to plane of metal, taken as 0.9 *h*.

q=Number of square inches of metal required in beam, in width of 12 inches.

TABLE OF SPACING REQUIRED FOR DIFFERENT SIZES OF CORRUGATED BARS FOR GIVEN AREA OF METAL IN RECTANGULAR BEAMS ONE FOOT WIDE.

	OLD STYLE BAR.					NEW STYLE BAR.							
C to C of Bar	½" BAR	¾" BAR	⅞" BAR	1" BAR	1¼" BAR	¼" BAR	1/3" BAR	½" BAR	⅝" BAR	¾" BAR	⅞" BAR	1" BAR	1¼" AR
2"	1.08□"	2.22□"	3.30□"	4.20□"	6.43□"	0.36□"	0.66□"	1.50□"	2.34□"	3.36□"	4.62□"	6.00□"	9.37□"
2½"	0.86□"	1.78□"	2.65□"	3.36□"	5.14□"	0.29□"	0.53□"	1.20□"	1.87□"	2.69□"	3.70□"	4,80□"	7.50□"
3"	0.72□"	1.48□"	2.20□"	2.80□"	4.28□"	0.24□"	0.44□"	1.00□"	1.56□"	2.24□"	3.08□"	4.00□"	6.24□"
3½"	0.62□"	1.27□"	1.89□"	2.40□"	3.67□"	0.21□"	0.38□"	0.86□"	1.34□"	1.92□"	2.64□"	3.43□"	5.36□"
4"	0.54□"	1.11□"	1.65□"	2.10□"	3.21□"	0.18□"	0.33□"	0.75□"	1.17□"	1.68□"	2.31□"	3.00□"	4.68□"
4½"	0.48□"	0.99□"	1.47□"	1.86□"	2.85□"	0.16□"	0.29□"	0.67□"	1.04□"	1.49□"	2.05□"	2.67□"	4.16□"
5"	0.43□"	0.89□"	1.32□"	1.68□"	2.57□"	0.14□"	0.26□"	0.60□"	0.94□"	1.34□"	1.85□"	2.40□"	3.75□"
5½"	0.39□"	0.81□"	1.20□"	1.52□"	2.34□"	0.13□"	0.24□"	0.55□"	0,85□"	1.22□"	1.68□"	2.18□"	3.41□"
6"	0.36□"	0.74□"	1.10□"	1.40□"	2.14□"	0.12□"	0.22□"	0.50□"	0.78□"	1.11□"	1.53□"	2.00□"	3.12□"
6½"	0.33□"	0.68□"	1.02□"	1.29□"	1.97□"	0.11□"	0.20□"	0.46□"	0.72□"	1.03□"	1.42□"	1.85□"	2.88□"
7"	0.31□"	0.63□"	0.94□"	1.20□"	1.83□"	0.10□"	0.19□"	0.43□"	0.67□"	0.96□"	1.32□"	1.72□"	2.68□"
7½"	0.29□"	0.59□"	0.88□"	1.12□"	1.71□"	0.10□"	0.18□"	0.40□"	0.62□"	0.89□"	1.23□"	1.60□"	2.50□"
8"	0.27□"	0.55□"	0.82□"	1.05□"	1.60□"	0.09□"	0.17□"	0.38□"	0.59□"	0.84□"	1.15□"	1.50□"	2.34□"
8½"	0.25□"	0.52□"	0.77□"	0.99□"	1.51□"	0.08□"	0.16□"	0.35□"	0.55□"	0.79□"	1.09□"	1.42□"	2.20□"
9"	0.24□"	0.50□"	0.73□"	0.93□"	1.43□"	0.08□"	0.15□"	0.33□"	0.52□"	0.75□"	1.02□"	1.33□"	2.08□"
9½"	0.23□"	0.47□"	0.69□"	0.88□"	1.35□"	0 08□"	0.14□"	0.32□"	0.49□"	0.71□"	0.97□"	1.26□"	1.97□"
10"	0.22□"	0.44□"	0.66□"	0.84□"	1.28□"	0.07□"	0.13□"	0.30□"	0.47□"	0.67□"	0.92□"	1.20□"	1.87□"
11"	0.20□"	0.40□"	0.60□"	0.76□"	1.17□"	0.07□"	0.12□"	0.27□"	0.43□"	0.61□"	0.84□"	1.09□"	1.70□"
12"	0.18□"	0.37□"	0.55□"	0.70□"	1.07□"	0.06□"	0.11□"	0.25□"	0.39□"	0.56□"	0.77□"	1.00□"	1.56□"

The accompanying curves give a means of readily figuring the ultimate resisting moment of a beam reinforced with a certain ratio of reinforcement, and at the same time gives the unit stress on the extreme fiber in compression, and the unit stress in the steel. An example will illustrate:

Find the ultimate strength of a beam, 1:2:5 concrete, when $p = .0101$. From the curve $M_o = 480d^2$, $s = 55000$ and $f = 2500$. Should the beam be over reinforced, the unit stress in the steel will be less than 55000.

Taking $p = .01418$, $M_o = 597d^2$, while $s = 50000$.

For convenience, tables have been prepared which give the ultimate moment of resistance of beams 12 inches wide, of varying heights, and reinforced as stated.

ROCK CONCRETE; 1 : 3 : 6 MIX.

ULTIMATE RESISTING MOMENT OF REINFORCED CONCRETE BEAMS, 12″ WIDE; VARIOUS PERCENTAGES OF METAL.

Depth of Beam=h	0.2% Rein. $q=.002(bh)$ $M_o=1127\ h^2$	0.4% Rein. $q=.004(bh)$ $M_o=2148\ h^2$	0.6% Rein. $q=.006(bh)$ $M_o=3140\ h^2$	.7065% Rein. $q=.007065(bh)$ $M_o=3654\ h^2$	0.8% Rein. $q=.008(bh)$ $M_o=3810\ h^2$	1.0% Rein. $q=.01(bh)$ $M_o=4073\ h^2$	1.25% Rein. $q=.0125(bh)$ $M_o=4354\ h^2$	1.5% Rein. $q=.015(bh)$ $M_o=4568\ h^2$	1.8% Rein. $q=.018(bh)$ $M_o=4782\ h^2$
4″	18000	34000	50000	**58000**	61000	65000	70000	73000	76000
5″	28	54	78	**91**	95	102	109	114	120
6″	40	77	113	**131**	137	146	157	164	172
7″	55	105	154	**179**	186	200	214	224	234
8″	72	137	201	**234**	244	260	278	292	306
9″	91	174	254	**296**	308	330	352	370	387
10″	113	215	314	**365**	381	407	435	468	478
11″	136	260	380	**442**	461	493	527	553	579
12″	162	309	452	**526**	548	586	627	658	689
13″	190	363	531	**618**	644	688	736	772	809
14″	221	421	615	**716**	746	798	854	895	938
15″	253	484	706	**822**	857	916	980	1028	1075
16″	288	550	804	**935**	975	1043	1114	1170	1225
17″	325	621	907	**1056**	1101	1177	1258	1320	1382
18″	365	696	1018	**1184**	1235	1320	1410	1480	1550
19″	406	775	1135	**1318**	1375	1471	1571	1648	1725
20″	451	860	1256	**1462**	1524	1629	1741	1827	1915
22″	545	1040	1520	**1770**	1844	1972	2108	2212	2316
24″	649	1238	1810	**2103**	2195	2347	2508	2630	2555

UNIT STRESS IN STEEL AT ULTIMATE LOAD.

$S=$	55000	55000	55000	**55000**	51000	44000	385000	34000	30000

UNIT STRESS ON EXTREME FIBRE IN COMPRESSION AT ULTIMATE LOAD.

$f=$	1175	1580	1880	**2000**	2000	2000	2000	2000	2000

ROCK CONCRETE; 1 : 2 : 5 MIX.

ULTIMATE RESISTING MOMENT OF REINFORCED CONCRETE BEAMS, 12″ WIDE; VARIOUS PERCENTAGES OF METAL.

Depth of Beam=h	0.4% Rein. q=.004(bh) Mo=2187 h²	0.6% Rein. q=.006(bh) Mo=3178 h²	0.8% Rein. q=.008(bh) Mo=4140 h²	1.00% Rein. q=.010(bh) Mo=5083 h²	**1.1016% Rein. q=0.011(bh) Mo=5660 h²**	1.30% Rein. q=.013(bh) Mo=5832 h²	1.50% Rein. q=.015(bh) Mo=6065 h²	2.00% Rein. q=.02(bh) Mo=6542 h²	2.75% Rein. q=.0275(bh) Mo=7047 h²
4″	35000	51000	66000	81000	**88000**	93000	97000	109000	112000
5″	55	80	103	127	**138**	145	151	163	176
6″	78	114	149	183	**200**	210	218	235	254
7″	107	155	203	249	**272**	285	297	320	345
8″	140	203	265	325	**355**	373	388	418	451
9″	177	257	335	412	**450**	472	491	530	571
10″	218	318	414	508	**556**	583	606	654	705
11″	265	385	501	615	**672**	705	734	791	852
12″	315	458	596	732	**799**	840	873	942	1014
13″	370	537	700	859	**938**	985	1025	1105	1192
14″	428	622	812	997	**1088**	1143	1188	1282	1382
15″	492	715	932	1144	**1249**	1312	1365	1472	1586
16″	560	813	1060	1301	**1421**	1493	1553	1675	1805
17″	632	918	1196	1469	**1604**	1685	1752	1890	2035
18″	708	1029	1341	1646	**1798**	1890	1965	2120	2282
19″	790	1147	1495	1835	**2005**	2105	2189	2360	2542
20″	875	1271	1656	2037	**2220**	2332	2425	2615	2818
22″	1058	1537	2004	2460	**2687**	2823	2938	3165	3410
24″	1259	1830	2385	2922	**3196**	3360	3494	3765	4055

UNIT STRESS IN STEEL AT ULTIMATE LOAD.

S =	55000	55000	55000	55000	**55000**	49000	45000	37000	30000

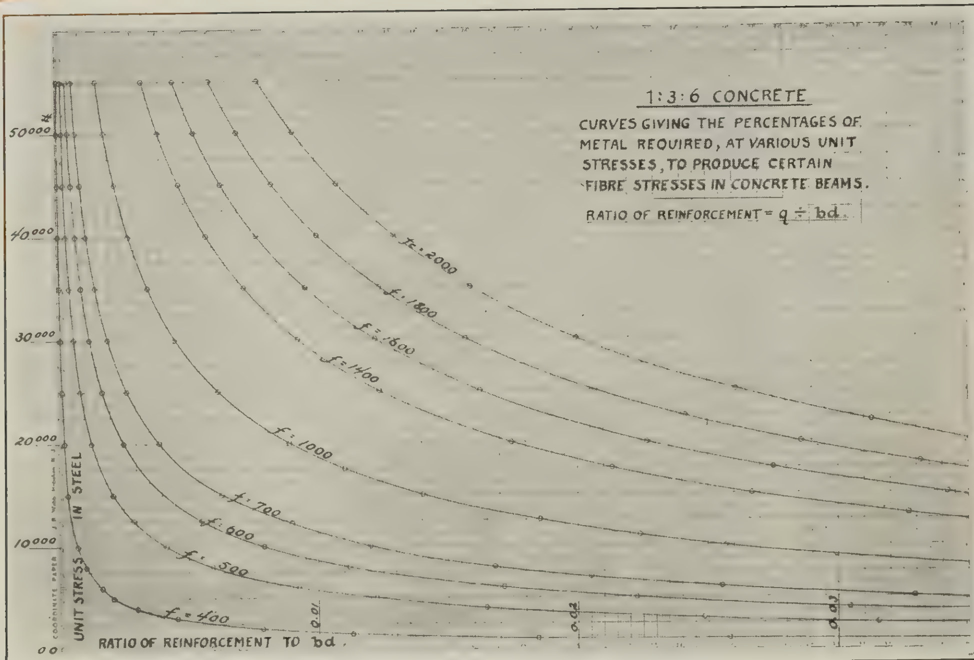
UNIT STRESS ON EXTREME FIBRE IN COMPRESSION AT ULTIMATE LOAD.

f=	1700	2080	2360	2600	**2700**	2700	2700	2700	2700

The formulæ as developed are not readily adapted to the solution of the general case, in which f and p are fixed, unless the corresponding s is known. Curves have accordingly been drawn from which the value of s may be obtained, and the value substituted in the formulæ for solution.

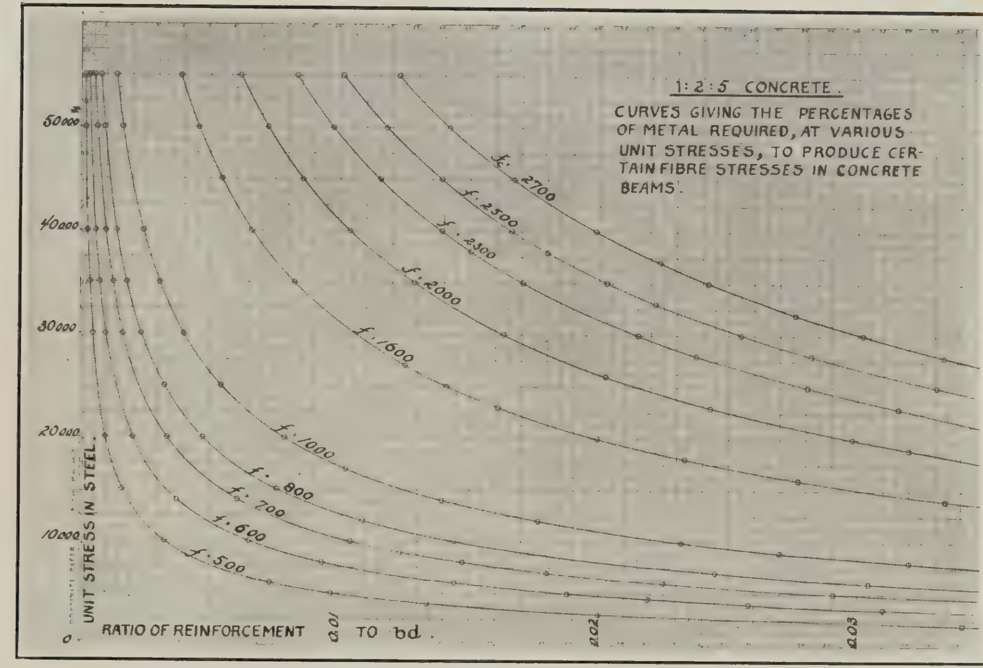
Example: A beam of 1:3:6 concrete has a ratio of reinforcement of .01. What stress in the steel will be required to develop 1,400 pounds per square inch extreme fiber stress in the concrete? On table page 168 read from left to right until vertical marked 0.01 is reached, then upwards until curve, $f=1400$, is intersected, from which it is found that $s=28750$; similarly for any other case.

It is to be noted that a .7% reinforcement of steel with an elastic limit of 30,000 pounds per square inch will develop less than $\frac{2}{3}$ of the full strength of concrete in compression.

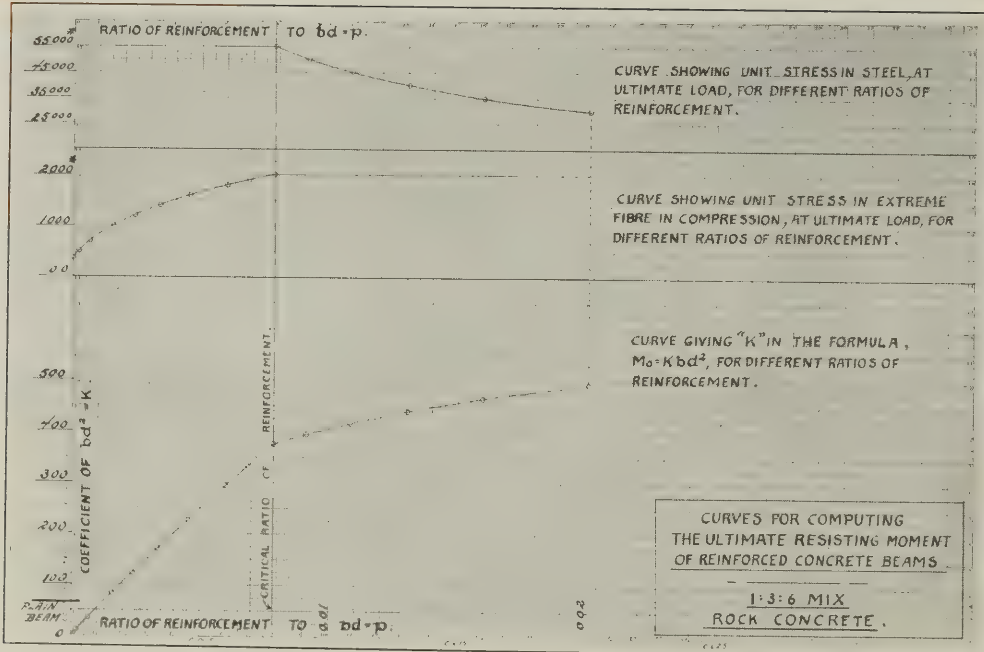

1:3:6 CONCRETE
CURVES GIVING THE PERCENTAGES OF METAL REQUIRED, AT VARIOUS UNIT STRESSES, TO PRODUCE CERTAIN FIBRE STRESSES IN CONCRETE BEAMS.
RATIO OF REINFORCEMENT = q ÷ bd.
UNIT STRESS IN STEEL
RATIO OF REINFORCEMENT TO bd.
50000 #
40000
30000
20000
10000
00
0.01
0.02
0.03
fc = 2000
f = 1800
f = 1600
f = 1400
f = 1000
f = 700
f = 600
f = 500
f = 400

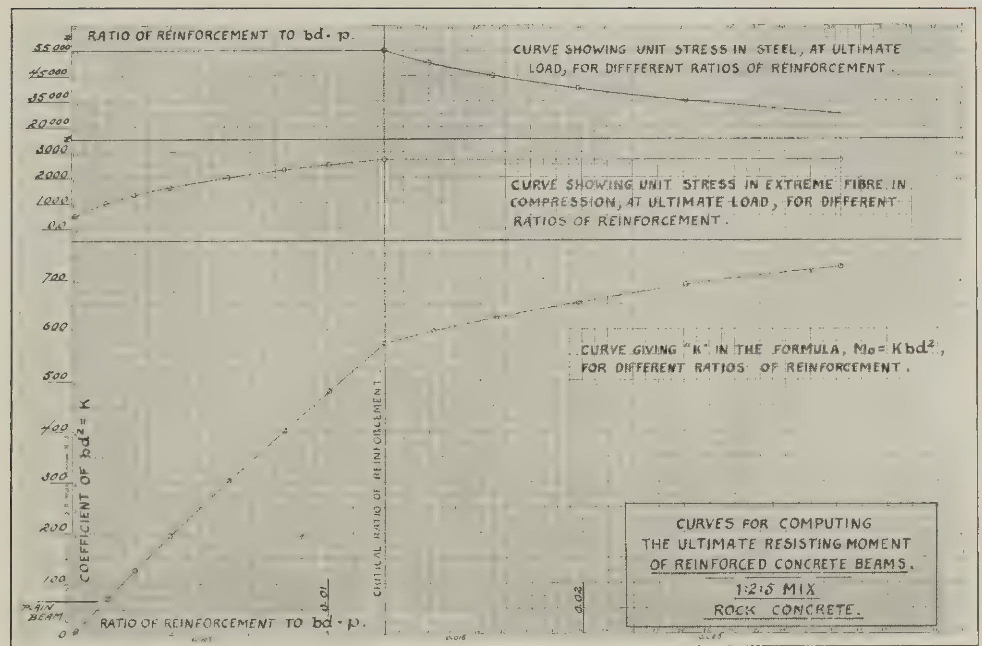
THE curves on pages 168 and 169 are not intended for use in designing, but are merely incorporated in the discussion to make it more complete mathematically.

A careful study of the foregoing discussion is absolutely necessary for the correct interpretation of these curves.



1:2:5 CONCRETE.
CURVES GIVING THE PERCENTAGES OF METAL REQUIRED, AT VARIOUS UNIT STRESSES, TO PRODUCE CERTAIN FIBRE STRESSES IN CONCRETE BEAMS.
f: 2700
f. 2500
f. 2300
f. 2000
f. 1600
f. 1000
f. 800
f. 700
f. 600
f. 500
50000 #
40000
30000
20000
10000
0
UNIT STRESS IN STEEL.
RATIO OF REINFORCEMENT TO bd.
0.01
0.02
0.03


RATIO OF REINFORCEMENT TO bd = p.
55000
45000
35000
25000
CURVE SHOWING UNIT STRESS IN STEEL, AT ULTIMATE LOAD, FOR DIFFERENT RATIOS OF REINFORCEMENT.
2000
1000
00
CURVE SHOWING UNIT STRESS IN EXTREME FIBRE IN COMPRESSION, AT ULTIMATE LOAD, FOR DIFFERENT RATIOS OF REINFORCEMENT.
CURVE GIVING "K" IN THE FORMULA, Mo = Kbd², FOR DIFFERENT RATIOS OF REINFORCEMENT.
500
400
300
200
100
PLAIN BEAM
0
COEFFICIENT OF bd² = K.
CRITICAL RATIO OF REINFORCEMENT.
0.01
0.02
RATIO OF REINFORCEMENT TO bd = p.
CURVES FOR COMPUTING THE ULTIMATE RESISTING MOMENT OF REINFORCED CONCRETE BEAMS
1:3:6 MIX
ROCK CONCRETE.


RATIO OF REINFORCEMENT TO bd · p.
55000
45000
35000
20000
3000
2000
1000
0.0
700
600
500
400
300
200
100
PLAIN BEAM
0
COEFFICIENT OF bd^2 = K
CRITICAL RATIO OF REINFORCEMENT
0.01
0.02
RATIO OF REINFORCEMENT TO bd · p.
CURVE SHOWING UNIT STRESS IN STEEL, AT ULTIMATE LOAD, FOR DIFFERENT RATIOS OF REINFORCEMENT.
CURVE SHOWING UNIT STRESS IN EXTREME FIBRE IN COMPRESSION, AT ULTIMATE LOAD, FOR DIFFERENT RATIOS OF REINFORCEMENT.
CURVE GIVING "K" IN THE FORMULA, $M_o = Kbd^2$, FOR DIFFERENT RATIOS OF REINFORCEMENT.
CURVES FOR COMPUTING THE ULTIMATE RESISTING MOMENT OF REINFORCED CONCRETE BEAMS.
1:2:5 MIX
ROCK CONCRETE.

REINFORCED CONCRETE BEAMS OF CIRCULAR OR ANNULAR SECTIONS

It is hoped that the following analysis and formulæ will be found useful in the design of chimneys, or to obtain the resisting moments of circular or annular sections.

In order to simplify the equations, the value of concrete in tension is neglected, and the modulus of elasticity of concrete is considered constant; these assumptions are justified on the ground that the results are sufficiently accurate for all practical purposes. Since the formulæ are meant to be used for working values of the stresses, the parabola representing the stress strain diagram will practically coincide with the tangent representing the initial modulus. Also, had the tension in the concrete been considered, which has a high value at working stresses, the per cent of steel so determined would have been very small and entirely inadequate to develop the compressive strength of the concrete at ultimate loading, when the effect of the tension on the concrete in resisting flexure is practically nil. By neglecting the tension, the factor of safety is made somewhat proportional to the working values chosen.

In addition to the above, the usual beam formulæ assumptions are made, such as invariability of plane sections, absence of initial stress, etc.

CIRCULAR BEAMS.

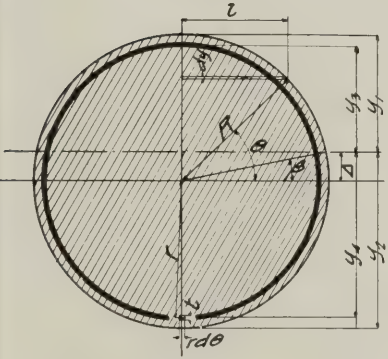


FIG. 1.
Section of Beam.

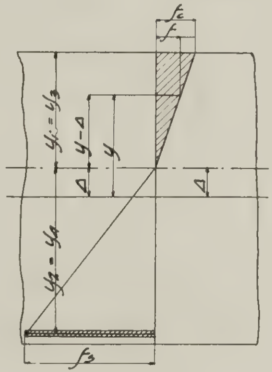


FIG. 2.
Stress Diagram.

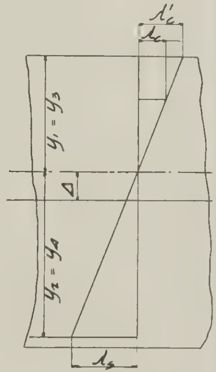


FIG. 3.
Strain Diagram.

Let figure (1) represent a circular section in which the steel is considered as a continuous shell of thickness t, and the neutral axis is at a distance Δ above the center of the section.

Let R=Radial distance to outside of beam.
r=Radial distance to center of reinforcement.
f_c=Extreme fibre stress in concrete.
f_s=Maximum stress in steel in tension.
y_1=Distance from neutral axis to extreme fiber in compression.
$y_2=2R-y_1$.
y_4=Distance from neutral axis to maximum stress in steel.
$y_3=2r-y_4$.
f=Stress at any point.
θ=Arc corresponding to ordinate y.
β=Arc corresponding to ordinate Δ.

Any elemental area parallel to the neutral axis can be expressed by ldy, where $l=\sqrt{R^2-y^2}$. If modulus is constant it follows that

$$f=f_c\frac{(y-\Delta)}{y_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

Elemental force $= fldy = \frac{f_c}{y_1}(y-\Delta)(R^2-y^2)^{\frac{1}{2}}dy$, then total force

$$P_c = 2\int_{\Delta}^{R} \frac{f_c}{y_1}(y-\Delta)(R^2-y^2)^{\frac{1}{2}}dy \quad \ldots\ldots\ldots\ldots (23)$$

$$= \frac{2f_c}{y_1}\left[\int_{\Delta}^{R} y(R^2-y^2)^{\frac{1}{2}}dy - \Delta\int_{\Delta}^{R}(R^2-y^2)^{\frac{1}{2}}dy\right]$$

Integrating and substituting limits.

$$P_c = \frac{2f_c}{y_1}\left[\tfrac{1}{3}(R^2-\Delta^2)^{\frac{3}{2}} - \frac{\pi R^2}{4}\Delta + \frac{\Delta^2}{2}(R^2-\Delta^2)^{\frac{1}{2}} + \frac{\Delta R^2}{2}\sin^{-1}\frac{\Delta}{R}\right] \ldots\ldots (24)$$

If $y_1 = ky_2$ and $y_1 + y_2 = 2R$,

Then $y_1 = \frac{2k}{1+k}R$, $y_2 = \frac{2R}{1+k}$, $\Delta = \frac{1-k}{1+k}R$, $k = \frac{R-\Delta}{R+\Delta}$

By substitution and reduction the total force in compression reduces to

$$P_c = \frac{R^2 f_c}{k}\left[\frac{8k^{\frac{3}{2}}}{3(1+k)^2} - \frac{\pi}{4}(1-k) + \left(\frac{1-k}{1+k}\right)^2 k^{\frac{1}{2}} + \frac{1-k}{2}\sin^{-1}\left(\frac{1-k}{1+k}\right)\right] \ldots\ldots (A)$$

As the expression inside the brackets is a constant for any value k, equation (A) reduces to the form

$$P_c = C_a f_c R^2$$

The moment of this force P_c about the neutral axis can be found by multiplying the elemental area by its lever arm $(y \dot{-} \Delta)$ and integrating:

Elemental moment

$$dm = f l dy (y - \Delta) = \frac{f_c}{y_1}(y-\Delta)^2 (R^2 - y^2)^{\frac{1}{2}} dy \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

By expansion

$$M_c=\frac{2f_c}{y_1}\left[\int_\Delta^R y^2(R^2-y^2)^{\frac{1}{2}}dy-2\Delta\int_\Delta^R y(R^2-y^2)^{\frac{1}{2}}dy+\right.$$

$$\left.\Delta^2\int_\Delta^R (R^2-y^2)^{\frac{1}{2}}dy\right] \quad \text{.................................} \quad (26)$$

Integrating and substituting limits.

$$M_c=\frac{2f_c}{y_1}\left[\left(\frac{R^2}{4}+\Delta^2\right)\left(\frac{\pi R^2}{4}-\frac{R^2}{2}\sin^{-1}\frac{\Delta}{R}\right)-\right.$$

$$\left.\Delta(R^2-\Delta^2)^{\frac{1}{2}}\left(\frac{2\Delta^2+13R^2}{24}\right)\right] \quad \text{.................................} \quad (27)$$

Substituting $y_1=\frac{2k}{1+k}R,\ \Delta=\left(\frac{1-k}{1+k}\right)R$

We have for the value of the moment of the force P_c about the neutral axis

$$M_c = \frac{R^3 f_c}{k(1+k)}\left[\frac{5-6k+5k^2}{4}\left(\frac{\pi}{4}-\frac{1}{2}\sin^{-1}\frac{(1-k)}{(1+k)}\right)-\right.$$

$$\left.\frac{(1-k)k^{\frac{1}{2}}}{12(1+k)^2}\left(15+22k+15k^2\right)\right] \qquad (B)$$

Which for a definite value of k reduces to the form

$$M_c = C_b f_c R^3$$

To find similar expressions for the steel, it will be found convenient to express the area element by $trd\theta$. Referring again to the figure

$$y = R\sin\theta, \text{ and } f = \frac{(y+\Delta)}{y_4} f_s$$

$$\text{Element of force} = \left(\frac{y+\Delta}{y_4}\right) f_s t r d\theta$$

Total force

$$P_s = \frac{2f_s tr}{y_4} \int_{-\beta}^{\frac{\pi}{2}} (r \sin \theta + \Delta)\, d\theta \qquad (28)$$

Integrating and substituting limits:

$$P_s = \frac{2f_s tr}{y_4} \left[r \cos\beta + \Delta \left(\frac{\pi}{2} + \beta \right) \right] \qquad (29)$$

$$\Delta = \frac{1-k}{1+k} r, \quad y_2 = \frac{2}{1+k} r, \quad \cos\beta = \frac{2k^{\frac{1}{2}}}{1+k}$$

from which it follows that the total force of tension in the steel is

$$P_s = f_s tr \left[2k^{\frac{1}{2}} + (1-k) \left(\frac{\pi}{2} + \sin^{-1} \frac{1-k}{1+k} \right) \right] \qquad (C)$$

or $P_s = C_c\, f_s tr.$

To find the moment, multiply area element by its distance from neutral axis $(y+\Delta.)$

$$dm_s = f_s \frac{(y+\Delta)^2}{y_4} tr d\theta \quad \text{.......... (30)}$$

$$M_s = 2\int_{-\beta}^{\frac{\pi}{2}} \frac{(y+\Delta)^2}{y_4} f_s tr d\theta \quad \text{.......... (31)}$$

Integrating and substituting limits—

$$M_s = \frac{2f_s tr}{y_4}\left[r^2\left(\frac{\pi}{4}+\frac{\beta}{2}-\frac{\sin 2\beta}{4}\right)+2\Delta r \cos\beta+\Delta^2\left(\frac{\pi}{2}+\beta\right)\right] \quad \text{.......... (32)}$$

Substituting for Δ and collecting terms

$$M_s = \frac{f_s tr^2}{(1+k)}\left\{\left(\frac{\pi}{2}+\sin^{-1}\frac{1-k}{(1+k)}\right)\left(\frac{3-2k+3k^2}{2}\right)+3k^{\frac{1}{2}}(1-k)\right\} \quad (D)$$

which for any given value of k reduces to the form

$$M_s = C_a f_s tr^2$$

Assigning values to f_c and f_s will determine the resisting moment, since $\frac{y_1}{y_4}=\frac{\lambda_c}{\lambda_s}$ will locate the neutral axis and equating P_c to F_s will determine the thickness t of the steel shell or the percentage of the reinforcement. The resisting moment is the sum of M_c and M_s for the proper values of k.

Example: The resisting moment of a 20″ circular beam is required. Allowable fiber stress in the concrete 700 lbs. per sq. in., assuming a class of concrete in which the corresponding deformation $\lambda_c=.00026$; and that the modulus of the steel is 29,000,000, we have

$\lambda_s=.00055$; it follows that $\frac{y_1}{y_4}=\frac{.00026}{.00055}=.473$

$$\frac{y_1}{y_4}=\frac{R-\Delta}{r+\Delta};\quad \Delta=\frac{R-.473r}{1+.473}$$

$$R=10'',\ r=8'' \qquad \Delta=4.21''$$

$$k_c=\frac{R-\Delta}{R+\Delta}=.408 \qquad k_s=\frac{r-\Delta}{r+\Delta}=.31$$

From table $k_c = .408$, $P_c = .31 f_c R^2$ and for $k_s = .31$, $F_s = 2.81 f_s t r$, $P_c = P_s$ then $.31 f_c R^2 = 2.81 f_s t r$, from which $t = .06$, or $\frac{1}{2}''$ corr. bars 4″ cts. may be used.

Resisting Moment.

From table for $k_c = .408$, $M_c = .106 f_c R^3$, for $K_s = .31$, $M_s = 3.05 f_s t r^2$. $M_r = M_c + M_s$, or, $0.106 f_c R^3 + 3.05 f_s t r^2 = 261700$, or practically 262,000 in lbs.

Resisting moment of an annular section is obtained by subtracting the values of P_c and M_s for the inner circle from those of the outer. Care being taken to use the proper values of k.

Example: The outside diam. of a chimney is 7′–0, inside diam. 5′–0 ft., determine resisting moment and reinforcement for $f_c = 700$ lbs. and $f_s = 16{,}000$ lbs. $R_1 = 42''$. $R_2 = 30''$ $r = 40''$

As in the previous example $\frac{y_1}{y_4}=.473$ from which

$$\Delta=\frac{42-.473\times 40}{1+.473}=15.7''$$

$$k_{R1}.=\frac{42-15.7}{42+15.7}=.456;\ \text{Corresponding } P_c=.347f_{c1}.R_1^2$$

$$k_{R2}=\frac{30-15.7}{30+15.7}=.31;\ \text{Corresponding } P_c=.228f_{c2}R_2^2$$

$$f_{c2}=\frac{30}{42}\times 700=500 \text{ lbs. per square inch.}$$

Total force in compression becomes
$.347f_{c1}R_1^2-.228f_{c2}R_2^2=325{,}800$ lbs.

$$k_s=\frac{40-15.7}{40+15.7}=.437,\quad P_s=2.434f_str$$

Total tension equals total compression $\therefore$ $325{,}800=2.43f_str$ or $t=.21$, or $\frac{7}{8}''$ bars 4″ cts. may be used.

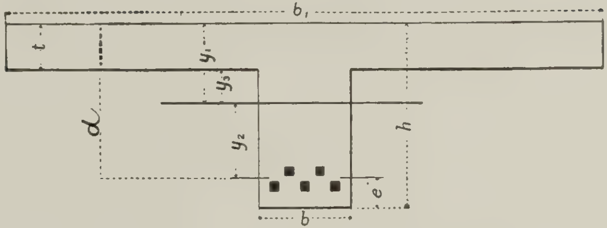
Resisting moment.
$M_{c1}=.128f_{c1}R_1^3=6{,}630{,}000$ in lbs.
$M_{c2}=.065f_{c2}R_2^3=877{,}500$ in lbs.
$M_s=2.63f_str^2=14{,}130{,}000$ in lbs.
$M_R=6{,}630{,}000-877{,}500+14{,}130{,}000=19{,}882{,}000$ in lbs.

This resisting moment is probably very much larger than would be required for such a stack, consequently the thickness of the concrete and the amount of reinforcement should be reduced until the resisting moment so obtained equals the external bending moment.

TABLE OF CONSTANTS FOR EQUATIONS A, B, C AND D FOR VARIOUS VALUES OF k.

k	C_a	C_b	C_c	C_d
.30	0.224	0.061	2.592	3.082
.35	.265	.082	2.531	2.902
.40	.305	.104	2.473	2.740
.45	.343	.124	2.420	2.591
.50	.380	.149	2.370	2.425
.60	.448	.196	2.279	2.222
.70	.511	.246	2.198	2.023
.80	.567	.296	2.125	1.850
.90	.519	.345	2.060	1.700
1.00	.667	.393	2.000	1.571

TEE-SHAPED BEAMS



LOCATION OF NEUTRAL AXIS.

Tee-shaped beams will be discussed only for the conditions existing at ultimate loading; the percentage of metal being such that the ultimate unit stresses in the concrete and steel are reached at the same time.

The tensional value of the concrete has been neglected.

In beams of Tee section y_1 is the same as for rectangular sections inasmuch as the position of the neutral axis is determined by the relative values of maximum compressibility of the concrete and extensibility of the steel inside the elastic limit or by the ratio of λ_c'' and λ_s.

We then have as before,

$$y_1 = \frac{E_s \lambda_c}{F + E_s \lambda_c} \qquad \text{(33)}$$

VALUES OF b_1 AND t.

Let S_v=Total shear in pounds along the two vertical planes of attachment between the wings and beam;

S_h=Total shear in pounds along the horizontal plane of attachment between the rib and floor plate;

σ=Maximum shearing strength of concrete in pounds per square inch:

$$K=\frac{y_3}{y_1}$$

l=Length of span in feet;

P'_c=Total compression in pounds at maximum load between neutral axis and underside of floor plate;

P''_c=Total compression in pounds in flange at maximum load.

All other functions as shown on cut, and in inches.

There are three methods of failure above the neutral axis:

1. By compression in the flange;
2. By deficiency in S_v owing to smallness of t;
3. By deficiency in S_h owing to smallness of b.

It would be desirable to have equal strength in all these directions, but this is not always possible, owing to other considerations. Where it is possible we have,

$$P_c'' = S_v = S_h \qquad (34)$$

But $S_h = 3b\sigma l$ .... (35)

and $S_v = 6t\sigma l$ .... (36)

The shearing stress is a maximum at the ends and for uniformly loaded beam varies uniformly to zero at the center. The value S_v may be increased about 50 per cent, owing to the metal reinforcement in the underside of floor plate which is always present in these designs, and placed in a direction at right angles to the tee beam. If vertical shear bars were used the same increase could be made in S_h, but ordinarily these would not be used, so we will not separately discuss this condition. Equation (36) then becomes

$$S_v = 9t\sigma l \qquad (37)$$

To get an expression for P_c''. We replace the stress strain diagram by a parabola with its vertex on the top of the beam, and coinciding with the stress strain diagram at this point and at the neutral axis; the area included by this parabola will closely approximate the actual stress strain area. By using this area we simplify the mathematics and get results sufficiently accurate for the tee beam discussion. We can then write

$$P_c'' = \frac{1}{3}(2 + K^3 - 3K^2) f_c b_1 y_1 \qquad (38)$$

This is on the assumption that the outer ends of the wings would be just as heavily stressed as the portion next to the beam. This would not be the case, the stress varying according to the ordinates to a parabola from zero at the outer ends to a maximum at the beam, and we should, therefore, multiply the above value by $\frac{2}{3}$. The portion of this width over the beam itself would not be subject to this modification, but there are other influences tending to offset this, so that the above is sufficiently correct.

Then $P_c'' = \frac{2}{9}(2+K^3-3k^2)\ f_c b_1 y_1$ .... (39)

From (35) and (37) we see that if t is not less than $\frac{b}{3}$, failure will not occur along the vertical sides of beam where wings attach. Now we will assume at once that t will not be allowed to have a value less than this. This leaves us to consider the relation between P_c'' and S_h only. We then have from (35) and (39)

$$3b\sigma l = \frac{2}{9}(2+K^3-3K^2)f_c b_1 y_1 \text{ from which}$$

$$b_1 = \frac{27b\sigma l}{2\ (2+K^3-3K^2)\ f_c y_1} \quad \text{.. (40)}$$

The theoretical relation between σ and f_c is

$$\sigma = \frac{f_c}{2 tan\theta} \text{(see Johnson's Materials of Construction, p. 29)........ ..... (41)}$$

where θ is the angle made by the plane of rupture on a compression specimen of moderate length with a plane at right angles to the direction of stress.

For concrete this angle is about 60°, hence

$$\sigma = \frac{f_c}{3.464} \quad \text{.........................} (42)$$

But this value is high in view of the liability of concrete to crack, and we recommend that twice the strength be provided in the shearing values on this basis that is used in compression.

We would then have $S_h = 2P_c''$ or

$$b_1 = \frac{27 b \sigma l}{4(2+K^3-3K^2) f_c y_1}$$ and substituting the value of σ

we have with sufficient accuracy,

$$b_1 = \frac{2bl}{(2+K^3-3K^2) y_1} \quad \text{.........................} (43)$$

We will now insert this value in (39) and proceed to obtain the moment of resistance. At times the above value of b_1 would be greater than the spacing of the beams, in which case the latter distance would be used for the value of b_1 in (39) and the other values worked over on this basis.

From (39) and 43) then we have,

$$P_c''=\frac{4}{9}f_c bl \quad\text{.........}\quad (44)$$

also $P_c'=f_c y_1 K^2(1-\frac{1}{3}K)b$ .... (45)

Then $P_c=P_c'+P_c''=\frac{4}{9}f_c bl+f_c y_1 K^2(1-\frac{1}{3}K)b$ .... (46)

$$P_s=Fq \quad\text{.........}\quad (47)$$

But $P_s=P_c$ .... (48)

From which

$$q=\frac{1}{F}\left[\frac{4}{9}f_c bl+f_c y_1 K^2(1-\frac{1}{3}K)\right] \quad\text{.........}\quad (49)$$

$$M_o=P_c'\tfrac{2}{3}Ky_1+P_c''\frac{(1+K)y_1}{2}+P_s y_2 \quad\text{.........}\quad (50)$$

Problem: Required the size of Tee-shaped beam necessary to carry a total ultimate load of 600 pounds per square foot on a span of 32 feet, ribs to be 9 feet apart.

Then $M=\frac{12\text{x}9\text{x}600\text{x}1024}{8}=8,300,000$ inch pounds.

For this spacing of beams the floor slab should be 4″ thick. We will assume $d=20''=y_1+y_2$.

Using good rock or gravel concrete, we have from (14)

$$y_1=.433\times20=8.66''; \text{ and } y_2=11.34$$

$$K=\frac{y_3}{y_1}=\frac{4.66}{8.66}=.538 \text{ and } K^2=.289$$

$$P_c'=f_c y_1 K^2(1-\tfrac{1}{3}K)b=2700\times8.66\times.289\times.82b= \quad 5500b$$

$$P_c''=\frac{4}{9}f_c bl=\frac{4}{9}\times2700\times32b \qquad =38400b$$

$$P_c=P_s=P_c'+P_c'' \qquad =43900b$$

Then $\quad q=\dfrac{43900}{55000}=.8b$

$$M_o=P_c'\times\tfrac{2}{3}Ky_1+P_c''\left(\frac{1+K}{2}\right)y_1+P_s y_2$$

$$=5500\times\tfrac{2}{3}\times4.66b+38400\times.769\times8.66b+43900\times11.34b$$
$$=17200b+256000b+498000b$$
$$=771200b$$

from which

$$b=\frac{8,300,000}{771200}=10.8''$$

Substituting in (43) we have

$$b_1=\frac{2bl}{(2+K^3-3K^2)y_1}=62''=5'-2''.$$

As this value of b_1, which we have used in determining the value of P_c'', is less than the spacing of the beams, we may use the beam as determined. It will be noted that t is greater than $\frac{b}{3}$.

From the foregoing we derive the following relations for a good grade of rock or gravel, 1:2:5 Portland cement concrete, where $f_c=2700$; $E_c=2,800,000$; $E_s=29,000,000$; $F=55000$.

$$P_c'=2700\, y_1K^2(1-\tfrac{1}{3}K)\,b$$

$$P_c''=1200bl$$

and $q=\dfrac{P_c'+P_c''}{55000}$ =number of square inches of metal required in rib.

$M_o=P_c'(\tfrac{2}{3}y_3+y_2)+P_c''\left(d-\dfrac{t}{2}\right)$ =ultimate moment of resistance in inch pounds.

All measures of length in inches except l, the length of span, which is in feet.

The value of t must be greater than one-third of b.

The value of b_1 represents the maximum width of flange that can be utilized in figuring the strength of the Tee, and its value is:

$$b_1 = \frac{2bl}{(2+K^3-3K^2)y_1}.$$

Where this value of b_1, exceeds materially the distance between the ribs, the above formulæ and the tables cannot be used, and a value of d will have to be chosen that will keep b_1 within its limit.

TABLE FOR THE DESIGN OF TEE BEAMS.

t	d	y_1	y_2	K	K^2	K^3	Area of Steel	Ultimate Moment	b_1
4″	10	4.33	5.67	.076	.0058	.0005	b(.0012+.0218 l)	b(390+ 9600 l)	.233 bl.
	11	4.76	6.24	.160	.0256	.0041	b(.0056+.0218 l)	b(2100+10800 l)	.218 bl.
	12	5.20	6.80	.231	.0534	.0123	b(.0126+.0218 l)	b(5270+12000 l)	208 bl.
	13	5.63	7.37	.289	.0835	.0241	b(.0208+.0218 l)	b(9720+13200 l)	.200 bl.
	14	6.06	7 94	.340	.1156	.0393	b(.0305+.0218 l)	b(15600+14400 l)	.195 bl.
	15	6.50	8.50	.385	.1482	.0571	b(.0412+.0218 l)	b(23100+15600 l)	.191 bl.
	17	7.37	9.63	.458	.2098	.0961	b(.0642+.0218 l)	b(41900+18000 l)	.185 bl.
	19	8.23	10.77	.513	.2631	.1350	b(.0881+.0218 l)	b(65450+20400 l)	.181 bl.
5″	12	5.20	6.80	.038	.0014	.0001	b(.0003+.0218 l)	b(135+11400 l)	.192 bl.
	13	5.63	7.37	.112	.0125	.0014	b(.0033+.0218 l)	b(1425+12600 l)	.180 bl.
	14	6.06	7.94	.175	.0306	.0054	b(.0086+.0218 l)	b(4080+13800 l)	.172 bl.
	15	6.50	8.50	.231	.0534	.0123	b(.0157+.0218 l)	b(8230+15000 l)	.167 bl.
	16	6.93	9.07	.279	.0778	.0217	b(.0240+.0218 l)	b(13680+16200 l)	.162 bl.
	18	7.80	10.20	.359	.1288	.0463	b(.0434+.0218 l)	b(28800+18600 l)	.155 bl.
	20	8.66	11.34	.423	.1789	.0757	b(.0653+.0218 l)	b(49500+21000 l)	.150 bl.
	22	9.53	12.47	.475	.2256	.1072	b(.0891+.0218 l)	b(76000+23400 l)	.147 pl.
6″	15	6.50	8.50	.077	.0059	.0005	b(.0018+.0218 l)	b(885+14400 l)	.155 bl.
	16	6.93	9.07	.134	.0179	.0024	b(.0058+.0218 l)	b(3100+15600 l)	.148 bl.
	18	7.80	10.20	.231	.0534	.0123	b(.0189+.0218 l)	b(11850+18000 l)	.139 bl.
	20	8.66	11.34	,307	.0943	.0289	b(.0360+.0218 l)	b(25959+20400 l)	.133 bl.
	22	9.53	12.47	.370	,1369	.0506	b(.0561+.0218 l)	b(45800+22800 l)	.129 bl.
	24	10.40	13.60	.423	.1789	.0757	b(.0785+.0218 l)	b(71400+25200 l)	.125 bl.
	26	11.26	14.74	.467	.2180	.1018	b(.1015+.0218 l)	b(102000+27600 l)	.123 bl.
	28	12.15	15.85	.507	.2570	.1303	b(.1275+.0218 l)	b(140000+30000 l)	.121 bl.

Note—The value of t must be greater than $\frac{1}{3}$b and there must be metal reinforcement in slab at right angles to beam.

SHEAR IN REINFORCED CONCRETE BEAMS

Let M_1=moment of resistance in inch pounds at 12″ from end of beam carrying its ultimate load.

M_0=ultimate moment of resistance in inch pounds at center.

l=span of beam in feet.

λ_2=elongation per inch at the plane of the metal, at section 12″ from end.

b=width of beam in inches.

σ=ultimate shearing strength of the concrete, about one-fourth the ultimate compressive strength.

Other functions as shown on pages 153 and 154.

Then $M_1=\dfrac{4l-4}{l^2}M_0$ for uniformly loaded beam............(1)

$$\lambda_2=\cfrac{M_1}{\cfrac{E_c\,b y_1^2}{3y_2}+\cfrac{E_c\,b y_2^2}{3}+\cfrac{E_s\,a^2 b y_2}{d}} \qquad (2)$$

$$b y_1^2=b y_2^2+\frac{2\,E_s\,a^2 b}{E_c\,d}y_2 \qquad (3)$$

$$y_1=h-y_2-e \qquad (4)$$

$$P_s=\frac{E\,\lambda_2 a^2 b}{d} \qquad (5)$$

After designing the beam by the beam formulæ, pages (159) and (160) $\dfrac{a^2 b}{d}$ y_1+y_2, E_c, E_s, and b are known. From (1) we obtain M_1 and from (3) and (4) y_1 and y_2. From (2) will be obtained λ_2, which inserted in (5) will give the pull

in the bars which has to be absorbed by shearing stress in the concrete over an area=$12b$. As it is desirable to take twice the factor of safety in shear that is taken in bending, P_s should not exceed $6b\sigma$, where σ is taken at one-fourth the compressive strength of the concrete.

If beams are loaded at two points some distance apart the maximum shearing stress is likely to be of a very different character. The bending moment being uniform between the loading points, the first cracks on the tension flange are as apt to occur under one of the loads as in the middle, and this will greatly reduce the strength of the anchorage of the ends of the bars represented by the shearing resistance of the concrete along the plane just above the metal between the crack and the end of the beam. This is especially true, as the maximum shearing stress along this plane is likely to be double the average stress. In such cases, as also in cases of uniform load where the shear exceeds the limits above given, the bars should be bent up at the ends, as shown in Figs. (1) and (2).

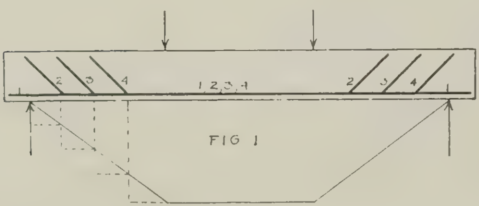


FIG 1

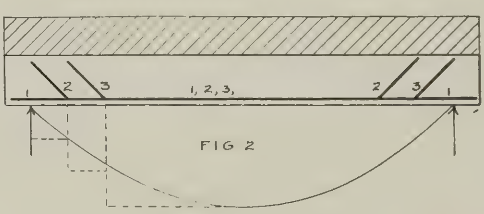


FIG 2

FLOOR PANELS

The foregoing discussion applies to beams on knife edge supports. Rectangular beams when incorporated in floor panels will have just about twice the capacity given by the formula, and the following tables, I to VI, are made on this basis.

To give a scientific discussion of this is almost impossible. It is a matter of actual practical experience. We can, however, see that it is reasonable to expect about such an increase. The haunches built down upon the lower flange of the supporting beams give a continuous girder action such as reduces the external bending moment one-third. Also the floor in adjacent panels produces an interior arching action, increasing the area of this compressive stress diagram about one-third, the effect of the two being to double the moment of resistance.

If the beam does not have the haunches projecting below as described, but is itself the full depth throughout, then we would add one-third only to the value of the moment of resistance.

Beams of Tee shape are not greatly strengthened by incorporation in floor panels, inasmuch as most of the compressive strength comes from the flanges, too high up to be affected by the interior arching action. That is to say, P_c'' (see page 186) would remain practically the same and P_c' would be increased probably 50 per cent. But the latter is usually so small as to make this increase of little value.

TABLE I.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS WITH No. 16GA. 2½″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_o''=Floor-Slab Moment of Resistance $=2\,M_o$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	680	0.68	435	0.54	300	0.45	....	...	....	....	....	....	....	...	16300
2½	1060	1.06	680	0.85	470	0.77	345	0.61	...	....	....	....	....	....	25460
3	1360	1.36	870	1.09	605	0.91	445	0.78	340	0.68	....	....	....	....	32830
3½	1640	1.64	1050	1.31	725	1.09	535	0.94	410	0.82	325	0.73	....	....	39240
4	1900	1.90	1220	1.52	845	1.27	620	1.09	475	0.95	380	0.85	305	0.76	45700
4½	2180	2.18	1390	1.74	970	1.45	710	1.24	545	1.09	430	0.97	350	0.87	52200
5	2450	2.45	1560	1.96	1090	1.63	795	1.40	610	1.22	485	1.09	390	0.98	58750
5½	2740	2.74	1740	2.17	1210	1.81	890	1.55	680	1.36	540	1.21	440	1.09	65300
6	3000	3.00	1910	2.39	1330	1.99	975	1.71	750	1.49	590	1.33	480	1.20	71900

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet.

Table II.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS WITH No. 10GA. 3″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_o''=Floor-Slab Moment of Resistance $=2 M_o$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	720	0.72	460	0.58	320	0.48	..	..	....	...	...	....	....	...	17350
2½	1130	1.13	730	0.91	505	0.76	370	0.65	....	....	....	....	....	...	27200
3	1620	1.62	1035	1.29	720	1.08	525	0.92	405	0.81	...	....	....	....	38800
3½	2140	2.14	1370	1.71	950	1.42	700	1.22	535	1.07	425	0.95	...	....	51300
4	2490	2.49	1595	1.99	1110	1.66	815	1.42	620	1.24	490	1.11	400	1.00	59800
4½	2860	2.86	1820	2.28	1270	1.90	930	1.62	710	1.42	565	1.26	455	1.14	68300
5	3200	3.20	2050	2.56	1430	2.13	1050	1.83	800	1.60	630	1.42	510	1.28	76900
5½	3560	3.56	2280	2.85	1580	2.37	1165	2.03	890	1.78	705	1.58	570	1.42	85500
6	3950	3.95	2520	3.14	1750	2.62	1280	2.24	980	1.96	775	1.74	630	1.57	94200

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet.

TABLE III.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS, USING ½″ SQUARE CORRUGATED STEEL BARS OF SUCH SPACING AS TO MAKE THE SLABS OF EQUAL STRENGTH IN TENSION AND COMPRESSION.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	Spacing of Bars in inches.	SPAN IN FEET.																		M_o''=Floor-Slab Moment of Resistance =2[M_o or M_o']
		8		9		10		11		12		13		14		15		16		
		U	C	U	C	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
3½	13	390	0.78	310	0.69	...	...	...	...	...	...	...	...	...	...	...	...	...	...	37500
4	11	550	1.09	430	0.97	350	0.87	...	...	...	...	...	...	...	...	...	...	...	...	52400
4½	9½	730	1.46	575	1.30	465	1.17	385	1.06	...	...	...	...	...	...	...	...	...	...	70000
5	8½	930	1.85	730	1.65	590	1.48	490	1.35	410	1.24	...	...	...	...	...	...	...	...	89000
5½	7½	1170	2.34	930	2.08	750	1.87	620	1.70	520	1.56	445	1.44	385	1.34	...	...	...	...	112400
6	7	1390	2.77	1090	2.46	885	2.21	735	2.02	615	1.84	525	1.71	455	1.58	395	1.48	...	...	133000
6½	6	1770	3.54	1400	3.15	1130	2.83	935	2.57	790	2.36	670	2.18	580	2.02	505	1.89	440	1.76	170000
7	5½	2100	4.21	1660	3.74	1350	3.37	1110	3.06	935	2.81	800	2.59	685	2.41	600	2.25	525	2.10	202000
7½	5	2500	5.00	1970	4.45	1600	4.00	1320	3.64	1110	3.34	945	3.08	815	2.86	710	2.67	625	2.50	240000

$U=\frac{M_o''}{1.5\, l^2}$ $C=\frac{M_o''}{6000\, l}$ l=span in feet. NOTE—Table is Based on Old Style Bars.

TABLE IV.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS WITH No. 16GA. 2½″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_o''=Floor-Slab Moment of Resistance $=2 M_o$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	930	0.93	595	0.75	415	0 62	...	...	...	...	...	...	...	...	22450
2½	1210	1.21	780	0 97	540	0.81	400	0.69	...	...	...	...	...	...	29200
3	1500	1.50	960	1.20	665	1.00	490	0.86	375	0.75	...	...	...	...	36000
3½	1780	1 78	1140	1.43	790	1.19	580	1.02	445	0.89	350	0.79	...	...	42850
4	2070	2.07	1330	1.66	920	1.38	675	1.18	520	1.03	410	0.92	330	0.83	49700
4½	2360	2.36	1510	1.89	1050	1.57	770	1.35	590	1.18	465	1.05	375	0.94	56600
5	2650	2.64	1690	2.12	1180	1.76	865	1.51	660	1.32	520	1.18	425	1.06	63500
5½	2930	2.93	1880	2.35	1300	1.96	960	1.67	735	1.47	580	1.30	470	1.17	70400
6	3220	3.22	2060	2.57	1430	2.15	1050	1.84	810	1.61	640	1.43	520	1.29	77300

$U=\frac{M_o''}{1.5 l^2}$ $C=\frac{M_o''}{6000 l}$ l=span in feet.

TABLE V.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS WITH No. 10GA. 3″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_o''=Floor-Slab Moment of Resistance $=2 M_o$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	1230	1.23	785	0.98	545	0.82	400	0.70	...	...	....	....	....	....	29500
$2\frac{1}{2}$	1600	1.60	1020	1.28	710	1.06	520	0.91	400	0.80	...	...	....	....	38100
3	1970	1.97	1260	1.58	875	1.32	645	1.13	495	0.99	390	0.88	....	....	47400
$3\frac{1}{2}$	2350	2.35	1500	1.88	1050	1.57	770	1.34	590	1.17	465	1.04	375	0.94	56450
4	2730	2.73	1750	2.18	1210	1.82	890	1.56	680	1.36	540	1.21	435	1.09	65500
$4\frac{1}{2}$	3110	3.11	1990	2.49	1380	2.07	1010	1.78	775	1.55	615	1.38	495	1.24	74700
5	3490	3.49	2230	2.79	1550	2.33	1140	1.99	875	1.74	690	1.55	560	1.39	83850
$5\frac{1}{2}$	3870	3.87	2480	3.10	1720	2.58	1265	2.21	970	1.94	765	1.72	620	1.55	93000
6	4260	4.26	2740	3.41	1900	2.84	1400	2.44	1070	2.14	840	1.90	680	1.71	102200

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet.

TABLE VI.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS, USING ½″ SQUARE CORRUGATED STEEL BARS OF SUCH SPACING AS TO MAKE THE SLABS OF EQUAL STRENGTH IN TENSION AND COMPRESSION.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

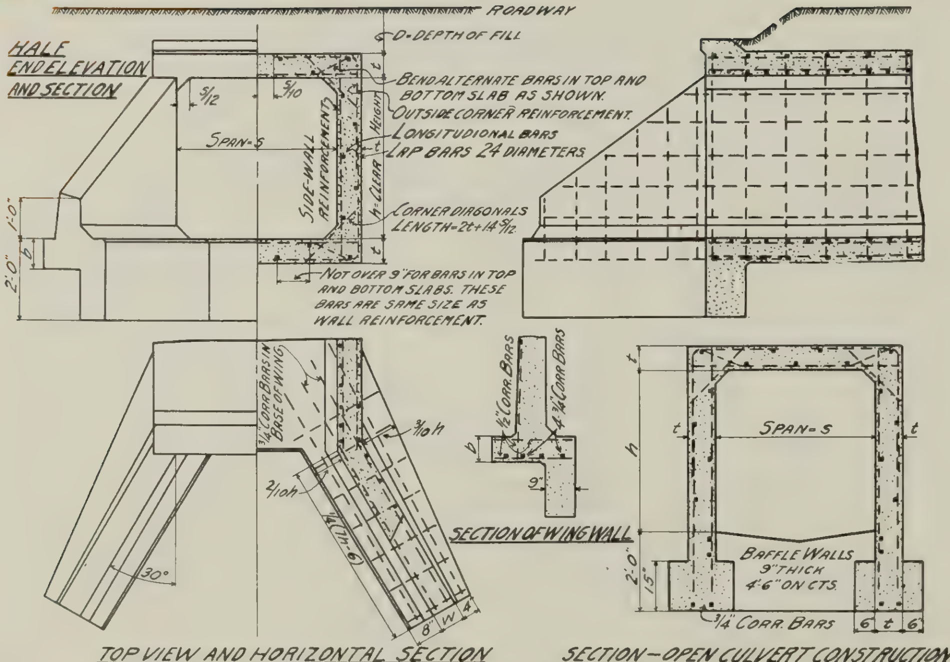
Thickness of Slab in inches.	Spacing of Bars in inches.	SPAN IN FEET. 8 U	8 C	9 U	9 C	10 U	10 C	11 U	11 C	12 U	12 C	13 U	13 C	14 U	14 C	15 U	15 C	16 U	16 C	M_o''=Floor-Slab Moment of Resistance $=2[M_o$ or $M_o']$
3½	7	775	1.55	610	1.38	495	1.24	410	1.13	....	....	....	....	...	...	....	....	..	....	74400
4	6	1070	2.14	840	1.90	685	1.71	565	1.56	475	1.43	405	1.32	....	....	....	....	....	....	102700
4½	5	1480	2.96	1165	2.63	945	2.36	780	2.15	660	1.97	560	1.82	480	1.69	420	1.58	....	....	142000
5	4½	1860	3.73	1470	3.31	1190	2.98	985	2.71	830	2.48	705	2.29	610	2.13	530	1.99	465	1.86	179000
5½	4	2340	4.68	1850	4.16	1500	3.75	1240	3.40	1040	3.12	885	2.88	765	2.68	665	2.50	585	2.35	225000
6	3½	2950	5.90	2330	5.25	1890	4.74	1560	4.30	1310	3.94	1120	3.65	965	3.38	840	3.15	740	2.96	284000
6½	3½	3250	6.50	2560	5.78	2080	5.20	1720	4.72	1440	4.34	1230	4.00	1060	3.71	920	3.46	810	3.24	311000
7	3	4100	8.24	3250	7.30	2630	6.58	2170	5.98	1830	5.48	1560	5.05	1340	4.70	1170	4.39	1030	4.12	395000
7½	3	4450	8.88	3500	7.88	2850	7.10	2350	6.45	1980	5.92	1680	5.46	1450	5.08	1260	4.75	1110	4.44	426000

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet. NOTE—Table Based on Old Style Bars.

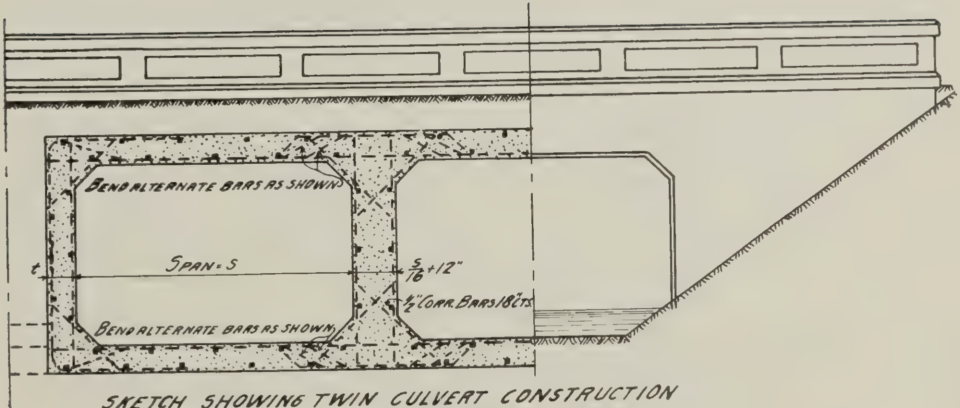
HIGHWAY CULVERTS

The following tables, in connection with the reference drawings, are meant to cover highway culverts up to 20′–0″ clear span, and with earth fill up to 12′–0″. The culverts have been arranged in three classes, according to the loadings for which they are intended. Class No. 1 is a light highway specification answering the purposes of ordinary county traffic where the heaviest load may be taken, as a 12-ton road roller. Class No. 2 is a heavy highway specification, designed for localities where heavy road rollers, up to 20 tons, and light electric cars, must be provided for. Class No. 3 is a city highway specification, designed for the heaviest interurban cars and should be used for all city work. These tables have been prepared especially for county engineers (and others interested in highway work), so that a design and a close estimate might be quickly made. The quantities of both

steel and concrete required per lineal foot of culvert are given in the tables, and the materials required for the wing walls may be obtained from the reference drawings. The stresses to which the culverts may be subjected have been carefully analyzed and the reinforcement so distributed that a permanent and satisfactory structure is insured. The concrete for this work should be of the best quality of rock or gravel concrete mixed in about the proportion 1:2½:5. No crushed rock or gravel should be used for slabs less than 9″ thick, that will not pass a ¾″ screen. The style of culvert to be used at a particular location, whether of the box or open type, will depend upon the conditions. For a soft ground, or one of uncertain character, the box type is desirable, but when a substantial foundation may be secured, with little danger from scour, the open culvert may be used. The concrete required for baffle walls is not included in tables.



Details Standard Highway Culverts.



SKETCH SHOWING TWIN CULVERT CONSTRUCTION
SLAB AND WALL REINFORCEMENT AS PER TABLES

CORR-BAR

CULVERT DATA FOR CLEAR SPAN OF 4'-0".

d=depth of fill. h=height of culvert. t=thickness of concrete.			BOX CULVERTS. Top and Bottom Reinforcement			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.		OPEN CULVERTS. Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds	Concrete cu. ft.	Steel Pounds
CLASS NO. 1. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														**CLASS NO. 1.**	
2'	2'	5 "	½"	8 "	4'-7"	½"	16"	3'-9"	½"	16 "	2'-7"	5.9	50	8.0	46
4'	3'	5 "	½"	8 "	4'-7"	½"	16"	4'-3"	½"	16 "	3'-7"	6.8	52	8.8	48
6'	4'	5 "	½"	8 "	4'-7"	½"	16"	4'-9"	½"	16 "	4'-7"	7.6	54	9.6	50
8'	5'	5½"	½"	7 "	4'-7"	½"	14"	5'-3"	½"	14 "	5'-7"	9.4	62	11.3	58
10'	6'	6 "	½"	6 "	4'-9"	½"	12"	5'-9"	½"	9 "	6'-8"	11.2	78	13.1	74
12'	7'	7½"	½"	6 "	4'-9"	½"	12"	6'-8"	½"	9 "	7'-8"	15.5	82	17.1	77
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														**CLASS NO. 2.**	
2'	2'	6 "	½"	7 "	4'-7"	½"	14"	3'-9"	½"	14 "	2'-7"	7.2	53	9.1	50
4'	3'	6 "	½"	7 "	4'-7"	½"	14"	4'-3"	½"	14 "	3'-7"	8.2	56	10.1	53
6'	4'	6 "	½"	7 "	4'-7"	½"	14"	4'-9"	½"	14 "	4'-7"	9.2	59	11.1	55
8'	5'	6 "	½"	7 "	4'-9"	½"	14"	5'-3"	½"	14 "	5'-7"	10.2	62	12.1	58
10'	6'	7 "	½"	6 "	4'-9"	½"	12"	5'-9"	½"	9 "	6'-8"	13.3	78	15.0	74
12'	7'	7½"	½"	5½"	4'-9"	½"	11"	6'-9"	½"	8¼"	7'-8"	15.5	91	17.2	82
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CARS.														**CLASS NO. 3.**	
2'	2'	6 "	½"	7 "	4'-9"	½"	14"	3'-9"	½"	14 "	2'-7"	7.2	54	9.1	50
4'	3'	6 "	½"	7 "	4'-9"	½"	14"	4'-3"	½"	14 "	3'-7"	8.2	57	10.1	54
6'	4'	6 "	½"	7 "	4'-9"	½"	14"	4'-9"	½"	14 "	4'-7"	9.2	60	11.1	56
8'	5'	6½"	½"	6 "	4'-9"	½"	12"	5'-3"	½"	12 "	5'-7"	11.2	70	13.0	65
10'	6'	7½"	½"	5½"	4'-9"	½"	11"	6'-2"	½"	8¼"	6'-8"	14.3	86	15.9	80
12'	7'	8 "	½"	5 "	4'-9"	½"	10"	6'-9"	½"	7½"	7'-8"	16.7	98	18.2	88

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 6′-0″.

BOX CULVERTS.														OPEN CULVERTS.	
d=depth of fill. h=height of culvert. t=thickness of concrete.			Top and Botton Reinforcement.			Outside Corner Reinforcement.			Side Walls, Reinforcement.			Quantities per Lineal Foot.		Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft	Steel pounds	Concrete cu. ft	Steel Pounds
CLASS NO. I. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														CLASS NO. I.	
2′	2′	6 ″	½″	7 ″	6′-8″	½″	14″	5′-0″	½″	14″	2′-7″	9.5	70	10.3	57
4′	3′	6 ″	½″	7 ″	6′-8″	½″	14″	5′-6″	½″	14″	3′-7″	10,5	74	11.3	60
6′	4′	7 ″	½″	6 ″	6′-8″	½″	12″	6′-0″	½″	12″	4′-7″	13.5	84	14.0	70
8′	5′	8 ″	½″	5 ″	7′-0″	½″	10″	6′-8″	½″	10″	5′-0″	17.0	100	17.0	85
10′	6′	9 ″	⅝″	6½″	7′-0″	⅝″	13″	7′-2″	⅝″	13″	7′-0″	21,0	136	20.4	113
12′	7′	9½″	⅝″	6 ″	7′-2″	⅝″	12″	7′-10″	⅝″	12″	8′-0″	23.6	148	23.1	125
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40 TON CAR.														CLASS NO. 2.	
2′	2′	7½″	½″	6 ″	6′-8″	½″	12″	5′-0″	½″	12″	2′-8″	12.1	76	12.3	63
4′	3′	7½″	½″	6 ″	6′-8″	½″	12″	5′-6″	½″	12″	3′-8″	13.4	80	13.6	67
6′	4′	7½″	½″	5 ″	6′-8″	½″	10″	6′-0″	½″	10″	4′-8″	14.6	95	14.8	78
8′	5′	8 ″	½″	5 ″	7′-0″	½″	10″	6′-8″	½″	10″	5′-10″	17.0	100	17.0	85
10′	6′	9 ″	⅝″	6½″	7′-0″	⅝″	13″	7′-2″	⅝″	13″	7′-0″	21.0	136	20.4	113
12′	7′	10 ″	⅝″	6 ″	7′-2″	⅝″	12″	7′-10″	⅝″	12″	8′-0″	25.0	148	24.2	125
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														CLASS NO. 3.	
2′	2′	8 ″	½″	5 ″	7′-0″	½″	10″	5′-0″	½″	10″	2′-10″	13.0	88	13.0	70
4′	3′	8 ″	½″	5 ″	7′-0″	½″	10″	5′-6″	½″	10″	3′-10″	14.4	92	14.3	75
6′	4′	8 ″	½″	5 ″	7′-0″	½″	10″	6′-0″	½″	10″	4′-10″	15.7	96	15.6	80
8′	5′	8½″	⅝″	7 ″	7′-0″	⅝″	14″	6′-8″	⅝″	14″	5′-10″	18.1	125	17.9	100
10′	6′	9½″	⅝″	6½″	7′-2″	⅝″	13″	7′-2″	⅝″	13″	7′-00″	22.0	136	21.5	113
12′	7′	10½″	⅝″	5½″	7′-2″	⅝″	11″	7′-10″	⅝″	11″	8′-00″	26.3	165	25.3	132

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 8′-0″.

d=depth of fill; h=height of culvert; t=thickness of concrete.			BOX CULVERTS. Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.		OPEN CULVERTS. Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds	Concrete cu. ft.	Steel Pounds
CLASS NO. 1. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														**CLASS NO. 1.**	
2′	4′	7½″	½″	6 ″	8′-10″	½″	12″	6′-9″	½″	12 ″	4′-10″	17.5	97	16.3	76
4′	5′	8 ″	½″	5 ″	8′-10″	½″	10″	7′-3″	½″	10 ″	5′-10″	20.0	115	18.5	90
6′	6′	9 ″	⅝″	7 ″	9′-0″	⅝″	14″	8′-0″	⅝″	14 ″	7′-0″	24 1	150	22.1	115
8′	7′	10 ″	⅝″	6 ″	9′-2″	⅝″	12″	8′-9″	⅝″	12 ″	8′-2″	28.7	178	26.0	136
10′	8′	11 ″	¾″	8 ″	9′-4″	¾″	16″	9′-6″	⅝″	8 ″	9′-4″	33.6	207	30.5	165
12′	9′	12 ″	¾″	7 ″	9′-6″	¾″	14″	10′-0″	⅝″	7 ″	10′-6″	39.0	240	35.0	190
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														**CLASS NO. 2.**	
2′	4′	9 ″	⅝″	7 ″	9′-0″	⅝″	14″	7′-0″	½″	14 ″	5′-0″	21.2	120	19.0	88
4′	5′	9 ″	⅝″	7 ″	9′-0″	⅝″	14″	7′-6″	½″	14 ″	6′-0″	22.7	125	20.5	91
6′	6′	9½″	⅝″	6½″	9′-0″	⅝″	13″	8′-0″	⅝″	13 ″	7′-0″	25.6	157	23.3	120
8′	7′	10½″	⅝″	5½″	9′-4″	⅝″	11″	8′-9″	⅝″	11 ″	8′-3″	30.2	188	27.3	143
10′	8′	11½″	¾″	7 ″	9′-4″	¾″	15″	9′-6″	⅝″	7½″	9′-5″	35.2	218	31.6	180
12′	9′	12½″	¾″	7 ″	9′-6″	¾″	14″	10′-2″	⅝″	7 ″	10′-7″	40.6	222	36.4	192
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														**CLASS NO. 3.**	
2′	4′	9½″	⅝″	6½″	9′-0″	⅝″	13″	7′-0″	½″	13 ″	5′-0″	22.4	130	20.2	92
4′	5′	9½″	⅝″	6½″	9′-0″	⅝″	13″	7′-6″	½″	13 ″	6′-0″	24.0	134	21.8	96
6′	6′	9½″	⅝″	6½″	9′-0″	⅝″	13″	8′-0″	⅝″	13 ″	7′-0″	25.6	157	23.3	120
8′	7′	10½″	⅝″	5½″	9′-4″	⅝″	11″	8′-9″	⅝″	11 ″	8′-3″	30.2	188	27.3	143
10′	8′	11½″	¾″	7½″	9′-4″	¾″	15″	9′-6″	⅝″	7½″	9′-5″	35.2	218	31.6	180
12′	9′	13 ″	¾″	7 ″	9′-6″	¾″	14″	10′-4″	⅝″	7 ″	10′-7″	42.5	224	35.8	194

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 10′-0″.

BOX CULVERTS.															OPEN CULVERTS.	
d—depth of fill. h—height of culvert. t=thickness of concrete.			Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.			Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds		Concrete cu. ft.	Steel Pounds
CLASS NO. I. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.															**CLASS NO. I.**	
2′	5′	10 ″	⅝″	6 ″	11′-2″	⅝″	12″	8′-6″	½″	12″	6′- 2″	29.2	166		24.7	114
4′	6′	10 ″	⅝″	6 ″	11′-2″	⅝″	12″	9′-0″	½″	12″	7′- 2″	30.8	170		26.3	118
6′	7′	10½″	⅝″	5½″	11′-4″	⅝″	11″	9′-6″	⅝″	11″	8′- 2″	34.2	214		29.3	156
8′	8′	12 ″	⅝″	5 ″	11′-6″	⅝″	10″	10′-3″	⅝″	10″	9′- 4″	41.4	240		35.2	174
10′	9′	13 ″	¾″	6½″	11′-8″	¾″	13″	11′-0″	⅝″	13″	10′- 6″	47.2	263		40.2	188
12′	10′	14½″	¾″	6 ″	12′-0″	¾″	12″	11′-9″	⅝″	12″	11′-10″	55.4	295		47.3	208
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.															**CLASS NO. 2.**	
2′	5′	10½″	⅝″	6″	11′. 4″	⅝″	12″	8′-6″	½″	12″	6′- 2″	30.7	168		25.9	116
4′	6′	10½″	⅝″	6 ″	11′- 4″	⅝″	12″	9′-0″	½″	12″	7′- 2″	32.5	170		27.6	118
6′	7′	11½″	⅝″	5½″	11′- 6″	⅝″	11″	9′-7″	⅝″	11″	8′- 4″	37.6	216		32.0	158
8′	8′	12½″	⅝″	5 ″	11′- 8″	⅝″	10″	10′-3″	⅝″	10″	9′- 6″	43.3	242		36.8	178
10′	9′	13½″	¾″	6½″	11′ 10″	¾″	13″	11′-0″	⅝″	13″	10′- 8″	49.2	266		41.8	192
12′	10′	15 ″	¾″	6 ″	12′- 0″	¾″	12″	11′-9″	⅝″	12″	11′-10″	57.7	295		48.9	208
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.															**CLASS NO. 3.**	
2′	5′	10½″	⅝″	5½″	11′- 4″	⅝″	11″	8′-6″	½″	11″	6′- 2″	30.7	178		25.9	126
4′	6′	10½″	⅝″	5½″	11′- 4″	⅝″	11″	9′-0″	½″	11″	7′- 2″	32.5	183		27.6	130
6′	7′	11½″	⅝″	5 ″	11′- 6″	⅝″	10″	9′-7″	⅝″	10″	8′- 4″	37.6	234		32.0	168
8′	8′	12½″	¾″	7 ″	11′- 8″	¾″	14″	10′-3″	⅝″	14″	9′- 6″	43.3	244		36.8	174
10′	9′	13½″	¾″	6½″	11′.10″	¾″	13″	11′-0″	⅝″	13″	10′- 8″	49.2	266		41.8	192
12′	10′	15 ″	¾″	6 ″	12′- 0″	¾″	12″	11′-9″	⅝″	12″	11′-10″	57.7	295		48.9	208

NOTE—All Corrugated Bars are New Style.

CULVERT DATA FOR CLEAR SPAN OF 12′-0″.

d=depth of fill. h=height of culvert. t=thickness of concrete.

BOX CULVERTS.														OPEN CULVERTS.	
			Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.		Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds	Concrete cu. ft.	Steel Pounds
CLASS NO. I. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														**CLASS NO. I.**	
2′	5′	9½″	⅝″	6″	13′-2″	⅝″	12″	9′-4″	½″	12″	6′-2″	31.6	185	25.4	125
4′	6′	10½″	⅝″	5½″	13′-4″	⅝″	11″	10′-0″	½″	11″	7′-2″	36.6	207	29.6	140
6′	7′	12½″	¾″	7″	13′-8″	⅝″	7″	10′-10″	½″	7″	8′-6″	46.0	286	37.0	196
8′	8′	13½″	¾″	6½″	13′-10″	⅝″	6½″	11′-6″	½″	6½″	9′-8″	52.2	317	42.1	220
10′	9′	15½″	¾″	5½″	14′-2″	¾″	11″	12′-4″	⅝″	11″	11′-0″	62.8	345	50.9	235
12′	10′	17″	¾″	5″	14′-4″	¾″	10″	13′-2″	⅝″	10″	12′-2″	72.2	391	58.5	257
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														**CLASS NO. 2.**	
2′	5′	11½″	⅝″	5″	13′-6″	⅝″	10″	9′-8″	½″	10″	6′-4″	38.2	223	30.2	151
4′	6′	12″	⅝″	5″	13′-6″	⅝″	10″	10′-4″	½″	10″	7′-4″	42.0	226	33.5	154
6′	7′	13″	¾″	6½″	13′-8″	¾″	13″	10′-10″	⅝″	13″	8′-6″	47.8	280	38.4	193
8′	8′	14″	¾″	6″	13′-10″	¾″	12″	11′-6″	⅝″	12″	9′-8″	54.1	308	43.7	212
10′	9′	16½″	¾″	5½″	14′-4″	¾″	11″	12′-6″	⅝″	11″	11′-2″	67.3	351	54.0	238
12′	10′	17½″	¾″	5″	14′-6″	¾″	10″	13′-2″	⅝″	10″	12′-4″	74.6	395	60.3	260
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														**CLASS NO. 3.**	
2′	5′	12½″	¾″	7″	13′-8″	⅝″	7″	9′-10″	½″	7″	6′-6″	41.8	264	32.9	182
4′	6′	12½″	¾″	7″	13′-8″	⅝″	7″	10′-4″	½″	7″	7′-6″	43.9	275	34.0	194
6′	7′	13″	¾″	6½″	13′-8″	¾″	13″	10′-10″	⅝″	13″	8′-6″	47.8	280	38.4	193
8′	8′	14″	¾″	6″	13′-10″	¾″	12″	11′-6″	⅝″	12″	8′-8″	54.1	308	43.7	212
10′	9′	16½″	¾″	5½″	14′-4″	¾″	11″	12′-6″	⅝″	11″	11′-2″	67.3	351	54.0	238
12′	10′	17½″	¾″	5″	14′-6″	¾″	10″	13′-2″	⅝″	10″	12′-4″	74.6	395	60.3	260

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 14′-0″.

BOX CULVERTS.														OPEN CULVERTS.	
d=depth of fill. h height of culvert. t=thickness of concrete.			Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.		Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds.	Concrete cu ft.	Steel Pounds
CLASS NO. 1. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														CLASS NO. 1.	
2′	5′	10½″	5/8″	6 ″	15′-4 ″	5/8″	12 ″	10′-2 ″	½″	12″	6′-2″	39.0	205	29.9	137
4′	6′	12 ″	5/8″	5 ″	15′-6 ″	5/8″	10 ″	11′-0 ″	½″	10″	7′.4″	46.7	250	35.9	165
6′	7′	14 ″	3/4″	6½″	15′-10″	3/4″	13 ″	11′-10″	½″	13″	8′-8″	57.1	275	44.0	180
8′	8′	16 ″	3/4″	5½″	16′-2 ″	3/4″	11 ″	12′-8 ″	5/8″	11″	10′ 0″	68.5	372	52.7	247
10′	9′	18 ″	7/8″	7 ″	16′-6 ″	3/4″	10½″	13′-6 ″	5/8″	14″	11′-4″	80.7	397	62.4	260
12′	10′	20 ″	7/8″	6½″	16′-10″	3/4″	9¾″	14′-4 ″	5/8″	13″	12′-8″	94.0	440	72.9	290
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														CLASS NO. 2.	
2′	5′	13½″	3/4″	6½″	15′-10″	3/4″	13 ″	10′-10″	½″	13″	6′-8 ″	50.5	269	37.9	178
4′	6′	13½″	3/4″	6½″	15′-10″	3/4″	13 ″	11′-4 ″	½″	13″	7′-8 ″	52.8	275	40.2	182
6′	7′	15 ″	3/4″	6 ″	16′-0 ″	3/4″	12 ″	12′-0 ″	½″	12″	8′-10″	61.5	304	47.0	200
8′	8′	17 ″	7/8″	7 ″	16′-4 ″	3/4″	10½″	12′-10″	5/8″	14″	10′-2 ″	73.0	385	56.1	255
10′	9′	19 ″	7/8″	7 ″	16′-8 ″	3/4″	10½″	13′-8 ″	5/8″	14″	11′-6 ″	85.5	400	65.8	262
12′	10′	20 ″	7/8″	6 ″	15′-10″	3/4″	9 ″	14′-4 ″	5/8″	12″	12′-8 ″	94.0	473	72.9	310
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														CLASS NO. 3.	
2′	5′	14½″	3/4″	6 ″	16′-0 ″	3/4″	12 ″	11′-0 ″	½″	12″	6′-10″	54.5	296	40.6	190
4′	6′	14½″	3/4″	6 ″	16′-0 ″	3/4″	12 ″	11′-6 ″	½″	12″	7′-10″	57.0	302	43.1	195
6	7′	15 ″	3/4″	6 ″	16′-0 ″	3/4″	12 ″	12′-0 ″	½″	12″	8′-10″	61.5	309	47.0	200
8′	8′	17 ″	7/8″	7 ″	16′.4 ″	3/4″	10½″	12′-10″	5/8″	14″	10′-2 ″	73.0	385	56.1	255
10′	9′	19 ″	7/8″	7 ″	16′-8 ″	3/4″	10½″	13′-8 ″	5/8″	14″	11′-6 ″	85.5	400	65.8	262
12′	10′	20 ″	7/8″	6 ″	16′-10″	3/4″	9 ″	14′-4 ″	5/8″	12″	12′-8 ″	94.0	473	72.9	310

NOTE—All Bars Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 16′-0″.

d=depth of fill.
h=height of culvert.
t=thickness of concrete.

BOX CULVERTS.														OPEN CULVERTS.	
			Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.		Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds	Concrete cu. ft.	Steel Pounds
CLASS NO. 1. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														**CLASS NO. 1.**	
2′	6′	11½″	⅝″	5½″	17′-6″	⅝″	11″	11′-9″	½″	11″	7′-4″	49.5	255	36.9	165
4′	7′	13½″	¾″	6½″	17′-10″	¾″	13″	12′-8″	½″	13″	8′-8″	60.5	310	45.2	205
6′	8′	15½″	⅞″	7½″	18′-2″	¾″	11¼″	13′-6″	½″	15″	10′-0″	72.5	365	54.2	230
8′	9′	18″	⅞″	7″	18′-6″	¾″	10½″	14′-5″	⅝″	14″	11′-4″	87.7	435	65.9	280
10′	10′	20″	1″	8″	18′-10″	¾″	8″	15′-2″	⅝″	16″	12′-8″	101.5	530	76.6	340
12′	11′	22″	1″	7″	19′-2″	¾″	7″	16′-0″	⅝″	14″	14′-0″	116.0	615	88.0	395
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														**CLASS NO. 2.**	
2′	6′	15″	⅞″	8″	18′-0″	⅝″	8″	12′-4″	½″	16″	7′-10″	65.0	335	47.5	210
4′	7′	15″	⅞″	8″	18′-0″	⅝″	8″	12′-10″	½″	16″	8′-10″	67.5	340	50.0	220
6′	8′	17″	⅞″	7″	18′-4″	¾″	10½″	13′-9″	½″	14″	10′-2″	79.7	390	59.2	250
8′	9′	19″	⅞″	7″	18′-8″	¾″	10½″	14′-7″	⅝″	14″	11′-6″	92.8	440	69.4	285
10′	10′	21″	1″	7½″	19′-0″	¾″	7½″	15′-5″	⅝″	15″	12′-10″	107.0	565	80.4	360
12′	11′	23″	1″	7″	19′-4″	¾″	7″	16′-2″	⅝″	14″	14′-2″	122.0	620	92.3	400
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														**CLASS NO 3.**	
2′	6′	16½″	⅞″	7½″	18′-4″	¾″	11¼″	12′-8″	½″	15″	8′-2″	71.7	355	52.2	220
4′	7′	16½″	⅞″	7½″	18′-4″	¾″	11¼″	13′-2″	½″	15″	9′-2″	74.5	360	55.0	225
6′	8′	17″	⅞″	7″	18′-4″	¾″	10½″	13′-8″	½″	14″	10′-2″	79 7	390	59.2	250
8′	9′	19″	⅞″	7″	18′-8″	¾″	10½″	14′-7″	⅝″	14″	11′-6″	92.8	440	69.4	285
10′	10′	21″	1″	7½″	19′-0″	¾″	7½″	15′-5″	⅝″	15″	12′-10″	107.0	565	80.4	360
12′	11′	23″	1″	7″	19′-4″	¾″	7″	16′-2″	⅝″	14″	14′-2″	122.0	620	92.3	400

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 18′-0″.

d=depth of fill. h=height of culvert. t=thickness of concrete.

BOX CULVERTS.															OPEN CULVERTS.	
			Top and Bottom Reinforcement.			Outside Corner Reinforcement.			Side Walls Reinforcement.			Quantities per Lineal Foot.			Quantities per Lineal Foot.	
d.	h.	t.	Size.	Spac.	Length.	Size.	Spac.	Length.	Size.	Spac.	Length.	Concrete cu. ft.	Steel pounds		Concrete cu. ft.	Steel Pounds
CLASS NO. 1. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.															**CLASS NO. 1.**	
2′	6′	12½″	¾″	7½″	19′-8″	¾″	15 ″	12′-10	½″	15″	7′-6″	59.0	290		42.5	180
4′	7′	15 ″	¾″	6 ″	20′-0″	¾″	12 ″	13′-9	½″	12″	8′-10″	73.1	360		53.0	230
6′	8′	17½″	⅞″	7½″	20′-6″	¾″	11¼″	14′-8	½″	15″	10′-4″	89.0	400		64.4	250
8′	9′	20 ″	⅞″	6 ″	20′-10″	¾″	9 ″	15′-7	⅝″	12″	11′-8″	105.5	545		77.0	340
10′	10′	22 ″	1 ″	7 ″	21′-2″	1 ″	14 ″	16′-7	⅝″	14″	13′-0″	120.5	625		88.7	390
12′	11′	24 ″	1 ″	6½″	21′-6″	1 ″	13 ″	17′-5	⅝″	13″	14′-4″	136.5	685		100.8	430
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.															**CLASS NO. 2.**	
2′	6′	16 ″	⅞″	7½″	20′-2 ″	¾″	11¼″	13′-5″	½″	15″	8′-0″	75.8	380		53.6	230
4′	7′	16 ″	⅞″	7½″	20′-2 ″	¾″	11¼″	13′-11″	½″	15″	9′-0″	78.5	385		56.3	235
6′	8′	19 ″	⅞″	7½″	20′-8 ″	¾″	11¼″	14′-11″	½″	15″	10′-6″	97.0	405		70.0	255
8′	9′	20½″	1 ″	7½″	21′-0 ″	1 ″	15 ″	15′-10″	⅝″	15″	11′-10″	108.5	565		78.8	350
10′	10′	23 ″	1 ″	7 ″	21′-4 ″	1 ″	14 ″	16′-9″	⅝″	14″	13′-2″	126.5	635		92.7	395
12′	11′	25 ″	1 ″	6 ″	21′-8 ″	1 ″	12 ″	17′-7″	⅝″	12″	14′-6″	143.0	740		105.5	460
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.															**CLASS NO. 3.**	
2′	6′	17½″	⅞″	7½″	20′-6 ″	¾″	11¼″	13′-8″	½″	15″	8′-4″	83.0	390		58.6	235
4′	7′	17½″	⅞″	7½″	20′-6 ″	¾″	11¼″	14′-2″	½″	15″	9′-4″	85.9	395		61.5	240
6′	8′	19 ″	⅞″	6½″	20′-8 ″	¾″	9¾″	14′-11″	½″	13″	10′-6″	97.0	460		70.0	285
8′	9′	20½″	1 ″	7½″	21′-0 ″	1 ″	15 ″	15′-10″	⅝″	15″	11′-10″	108.5	565		78.8	350
10′	10′	23 ″	1 ″	7 ″	21′-4 ″	1 ″	14 ″	16′-9″	⅝″	14″	13′-2″	126.5	635		92.7	395
12′	11′	25 ″	1 ″	6 ″	21′-8 ″	1 ″	12 ″	17′-7″	⅝″	12″	14′-6″	143.0	740		105.5	460

NOTE—All Bars are Corrugated Bars, New Style.

CULVERT DATA FOR CLEAR SPAN OF 20′-0″.

d=depth of fill.
h=height of culvert.
t=thickness of concrete.

BOX CULVERTS.														OPEN CULVERTS,	
d.	h.	t.	Top and Bottom Reinforcement. Size.	Spac.	Length.	Outside Corner Reinforcement. Size.	Spac.	Length.	Side Walls Reinforcement. Size.	Spac.	Length.	Quantities per Lineal Foot. Concrete cu. ft.	Steel pounds	Quantities per Lineal Foot. Concrete cu. ft.	Steel Pounds
CLASS NO. I. LIGHT HIGHWAY SPECIFICATION. 12-TON ROAD ROLLER.														**CLASS NO. I.**	
2′	6′	13½″	¾″	6½″	21′-10″	¾″	13 ″	13′- 9″	½″	13″	7′- 8″	69.4	350	48.4	215
4′	7′	16½″	⅞″	7 ″	22′- 4″	¾″	10½″	14′- 9″	½″	14″	9′- 2″	87.4	440	61.4	270
6′	8′	19½″	⅞″	6½″	22′-10″	¾″	9¾″	15′- 9″	½″	13″	10′- 8″	107.0	490	75.6	300
8′	9′	22 ″	1 ″	7 ″	23′- 2″	1 ″	14 ″	16′-10″	⅝″	14″	12′- 0″	125.5	650	89.0	400
10′	10′	24 ″	1 ″	6½″	23′- 6″	1 ″	13 ″	17′- 8″	⅝″	13″	13′- 4″	141.5	710	101.5	445
12′	11′	26½″	1 ″	6 ″	22′-10″	1 ″	12 ″	18′- 7″	⅝″	12″	14′- 8″	162.0	790	116.5	490
CLASS NO. 2. HEAVY HIGHWAY SPECIFICATION. 20-TON ROLLER OR 40-TON CAR.														**CLASS NO. 2.**	
2′	6′	19 ″	⅞″	6½″	22′- 8″	¾″	9¾″	14′- 8″	½″	13″	8′- 6″	98.0	470	67.3	285
4′	7′	19 ″	⅞″	6½″	22′- 8″	¾″	9¾″	15′- 2″	½″	13″	9′- 6″	101.5	475	70.5	290
6′	8′	20½″	1 ″	7½″	23′- 0″	1 ″	15 ″	16′- 0″	½″	15″	10′-10″	113.0	555	79.7	340
8′	9′	23 ″	1 ″	7 ″	23′- 4″	1 ″	14 ″	17′- 0″	⅝″	14″	12′- 2″	131.5	655	93.3	405
10′	10′	25 ″	1 ″	6 ″	23′- 8″	1 ″	12 ″	17′-10″	⅝″	12″	13′- 6″	148.0	770	105.5	480
12′	11′	27 ″	1 ″	6 ″	24′- 0″	1 ″	12 ″	18′- 8″	⅝″	12″	14′-10″	165.5	795	119.0	495
CLASS NO. 3. CITY HIGHWAY SPECIFICATION. 60-TON STREET CAR.														**CLASS NO. 3.**	
2′	6′	19 ″	⅞″	6½″	22′- 8″	¾″	9¾″	14′- 8″	½″	13″	8′- 6″	98.0	470	67.3	285
4′	7′	19 ″	⅞″	6½″	22′- 8″	¾″	9¾″	15′- 2″	½″	13″	9′- 6″	101.5	475	70.5	290
6′	8′	20½″	1 ″	7½″	23′- 0″	1 ″	15 ″	16′- 0″	½″	15″	10′-10″	113.0	555	79.7	340
8′	9′	23 ″	1 ″	7 ″	23′- 4″	1 ″	14 ″	17′- 0″	⅝″	14″	12′- 2″	131.5	655	93.3	405
10′	10′	25 ″	1 ″	6 ″	23′- 8″	1 ″	12 ″	17′-10″	⅝″	12″	13′- 6″	148.0	770	105.5	480
12′	11′	27 ″	1 ″	6 ″	24′- 0″	1 ″	12 ″	18′- 8″	⅝″	12″	14′-10″	165.5	795	119.0	495

NOTE—All Bars are Corrugated Bars, New Style.

TESTS OF THE UNION BETWEEN CONCRETE AND STEEL

A recent issue of Beton and Eisen gave the results of a series of tests upon the holding power of different types of rods imbedded in concrete, made in the laboratories of the Massachusetts Institute of Technology by Prof. C. W. Spofford.

Portland cement concrete was used, made in the following proportions by weight: One part cement, three parts sand, six parts broken stone. This mixture was used in order that the results would correspond with tests upon beams and columns which were under way at the same time. The mixture, however, is very lean and would not again be used. The sand was clean, but rather coarse grained, containing approximately 47 per cent of voids. The broken stone was a mixture of two parts of 1″ trap and one part of ½″ trap. The mixing was thoroughly done by hand, the concrete being wet enough when tamped into the moulds to flush water to the surface. The moulds were, in some cases, not as tight as they should have been and some water leaked out, carrying with it some of the cement. It is not believed, however, that the loss thereby was sufficient to injure the results of the tests, except possibly in a very few cases. The rods were all thorougly cleaned by a sand blast, thus insuring uniformity in the surface conditions.

A 100,000-pound Olsen vertical testing machine was used, rigged with short uprights, carrying the platform upon which the specimens were placed. The load upon the bearing end of the concrete block was distributed by the interposition of a sheet of ½″ felt between the concrete and an annular steel ring resting upon the platform of the machine. In all cases the rod projected a short distance at the upper end of the block (the pull being downward at the lower end), and this projecting end was carefully watched in order to detect the first evidence of slipping. The rods used were round, square, flat, square but twisted through an angle of 20 degrees (Ransome rod), Thacher and Johnson. The table has been arranged from the original table in Beton and Eisen so that bars of the same size are together.—*Reprinted from the Railroad Gazette, for September 18, 1903.*

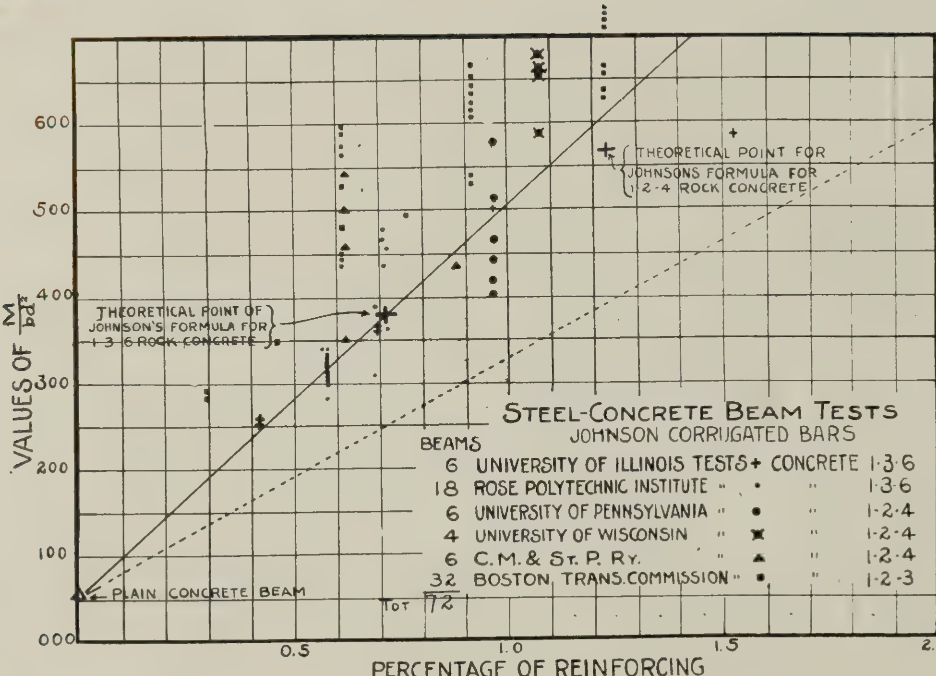
The following tables give the results of Prof. Spofford's tests, and also of some recent tests made by Prof. F. H. Constant of the University of Minnesota. In these latter tests it is interesting to note the high unit stresses obtained with deformed bars, and particularly with the Corrugated Bar, for the short imbedment used. This length of imbedment appears to be the proper one for the 1: 2: 4 concrete but not large enough for the leaner mixtures, making the reported values for the 1: 3: 6: and 1: 4: 8 concrete somewhat erratic.

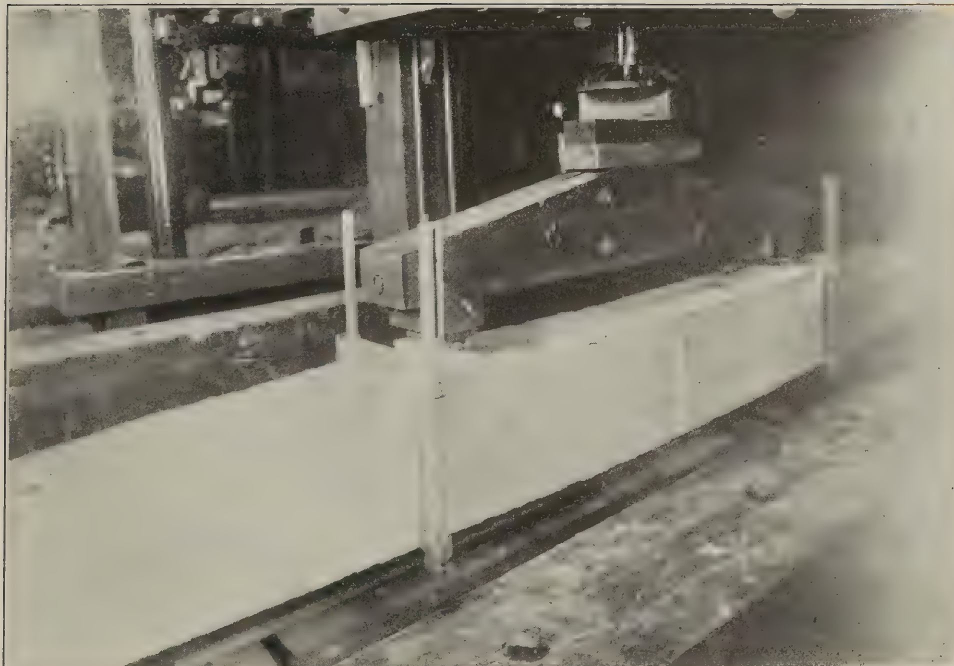
RESULTS OF TESTS BY PROF. SPOFFORD ON THE UNION BETWEEN CONCRETE AND STEEL.

No. of test.	Type of rod, inch.	Size of concrete block, inch.	Length of rod imbedded in concrete, inch.	Breaking load, pounds.	Minimum area of cross-section of rod, square inch.	Shearing stress in pounds per square inch of net section.	Stress on rod in pounds per square inch of net section.	Remarks.
1	Ransome 1-2	6x6	12	12,100	0.25	504	48,400	Concrete split longitudinally
4	Ransome 1-2	8x8	12	8,300	0.25	346	33,200	Rod slipped at 8,000, dropped to 6,000, rose again to 8,300, where concrete split. Rod pulled through 3 in.
13	Thacher 1-2	6x6	12	4,850	0.18	270	26,900	Rod slipped, concrete split
22	**Johnson 1-2**	**6x6**	**12**	**12,200**	**0.14**	**678**	**87,200**	Concrete split
2	Ransome 1-2	6x6	16	8,100	0.25	253	32,400	Concrete split longitudinally
5	Ransome 1-2	8x8	16	14,000	0.25	438	56,000	Rod slipped at 12,000, dropped to 8 000, rose again to 14,000, where concrete split. Rod pulled through 5 in.
14	Thacher 1-2	6x6	16	8,200	0.18	340	45,500	Rod slipped at 8,100, concrete split
23	**Johnson 1-2**	**6x6**	**16**	**13,120**	**0.14**	**545**	**93,700**	Concrete split
3	Ransome 1-2	6x6	26	16,800	0.25	323	67,200	Concrete crushed on end
6	Ransome 1-2	8x8	26	15,000	0.25	288	60,000	Rod slipped at 15,000, rod pulled through max. stress 14,400 Rod pulled through 11 1-2 in
15	Thacher 1-2	6x6	26	10,550	0.18	272	58,600	Rod broke
24	**Johnson 1-2**	**6x6**	**26**	**13,750**	**0.14**	**354**	**98,400**	Rod broke
7	Ransome 3-4	8x8	20	25,900	0.56	431	46,300	Rod slipped at 18,000, concrete split
16	Thacher 3-4	8x8	20	21,150	0.39	478	53,000	Rod slipped at 19,050, concrete split
25	**Johnson 3-4**	**8x8**	**20**	**27,600**	**0.31**	**619**	**89,100**	Concrete split
8	Ransome 3-4	8x8	24	31,900	0.56	443	57,000	Rod slipped at 14,000, concrete split
17	Thacher 3-4	8x8	24	18,300	0.39	344	45,900	Concrete split
26	**Johnson 3-4**	**8x8**	**24**	**25,000**	**0.31**	**467**	**80,600**	Concrete split
31	3-4 round	8x8	24	15,300	0.44	271	38,400	Rod slipped
34	3-4 square	8x8	24	19,700	0.56	274	35,200	Rod slipped
37	1 1-8 x 1-2	8x8	24	12,400	0.56	159	22,100	Rod slipped
40	1 1-2 x 3-8	8x8	24	20,300	0.56	226	36,300	Rod slipped
43	2 1-4 x 1-4	8x8	24	5,000	0.56	42	8,930	Rod slipped (specimen inj'd)
32	3-4 round	8x8	31	18,600	0.44	255	42,200	Rod slipped
35	3-4 square	8x8	31	22,600	0.56	243	40,400	Rod slipped
38	1 1-8 x 1-2	8x8	31	20,300	0.56	201	36,200	Rod slipped
41	1 1-2 x 3-8	8x8	31	21,700	0.56	188	38,800	Rod slipped
44	2 1-4 x 1-4	8x8	31	25,500	0.56	165	45,500	Rod slipped
9	Ransome 3-4	8x8	36	36,600	0.56	339	63,500	Concrete split
18	Thacher 3-4	8x8	36	23,700	0.39	297	59,400	Rod broke
27	**Johnson 3-4**	**8x8**	**36**	**28,000**	**0.31**	**483**	**90,500**	Concrete split
33	3-4 round	8x8	36	18,600	0.44	219	42,200	Rod slipped
36	3-4 square	8x8	36	23,900	0.56	221	42,700	Rod slipped
39	1 1-8 x 1-2	8x8	36	21,700	0.56	185	38,700	Rod slipped
42	1 1-2 x 3-8	8x8	36	22,130	0.56	164	39,500	Rod slipped
45	2 1-4 x 1-4	8x8	36	26,100	0.56	145	46,600	Rod slipped

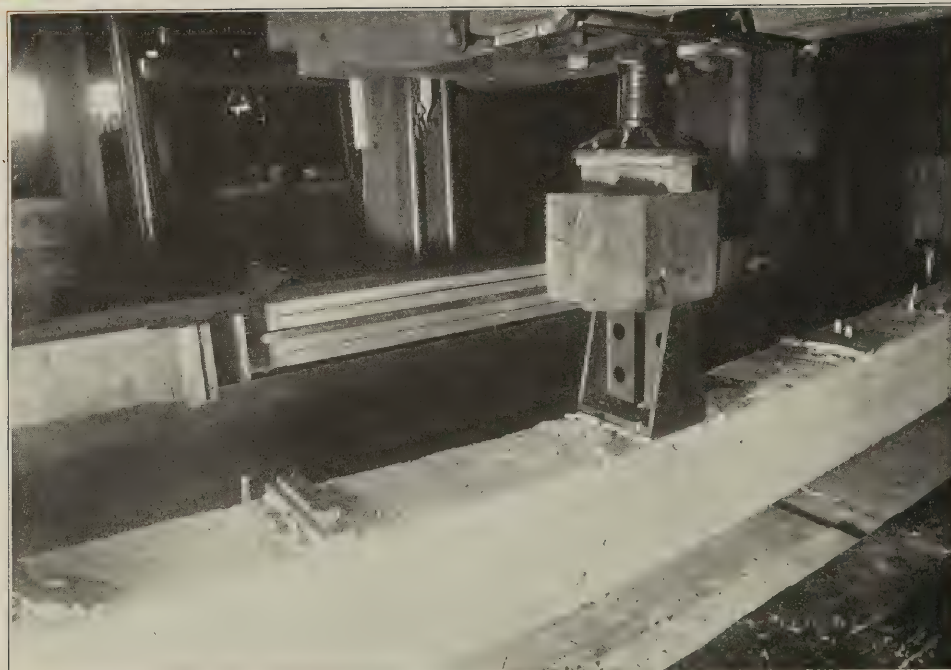
RESULTS OF TESTS BY PROF. F. H. CONSTANT ON BOND BETWEEN STEEL AND CONCRETE. TABLE OF COMPARATIVE MEAN VALUES.

No. of Tests.	Type of Bar.	Size.	Age of Conc.	Min. Net Section.	Super. Area per Lin. ft.	Imbedded Length.	Total Load.	Load per sq. in. Surface.	Unit Stress in Bar.	REMARKS.
					1: 2: 4 CONCRETE.					
3	**Johnson**	**¾**	**28**	**0.31**	**2.43**	**8.25**	**31,620**	**1,577**	**102,000**	**Concrete split.**
3	Thacher	¾	28	0.39	2.21	8.25	22,100	1,112	56,600	Bar broke.
3	Ransome	¾	28	0.56	3.00	8.20	24,470	994	43,700	Concrete split.
3	Truscon	¾	28	0.44	2.36	8.31	16,830	858	38,309	Rod slipped.
3	Round	¾	28	0.44	2.36	8.23	16,600	854	37,800	Rod slipped.
3	Round	⅜	28	0.11	1.18	8.48	4,580	454	41,600	Rod slipped.
3	Flat	1¼x⅜	28	0.47	3.25	8.30	10,630	394	22,600	Rod slipped.
3	Flat	2x¼	28	0.50	4.50	8.29	12,550	336	25,000	Rod slipped.
					1: 3: 6 CONCRETE.					
3	**Johnson**	**¾**	**28**	**0.31**	**2.43**	**8.62**	**12,360**	**591**	**39,900**	**Concrete split.**
3	Thacher	¾	28	0.39	2.21	8.37	10,330	559	26,600	Concrete split.
3	Ransome	¾	28	0.56	3.00	8.67	13,470	519	24,000	Concrete split.
3	Truscon	¾	28	0.44	2.36	8.77	8,630	417	19,600	Concrete split, rod slipped.
3	Round	¾	28	0.44	2.36	8.44	8,430	424	19,100	Rod slipped.
3	Round	⅜	28	0.11	1.18	8.12	3,530	368	32,100	Rod slipped.
3	Flat	1¼x⅜	28	0.47	3.25	8.10	6,070	230	12,900	Rod slipped.
3	Flat	2x¼	28	0.50	4.50	8.25	10,080	272	20,700	Rod slipped.
					1 : 4 : 8 CONCRETE.					
3	**Johnson**	**¾**	**28**	**0.31**	**2.43**	**8.21**	**18,120**	**908**	**58,500**	**Concrete split.**
3	Thacher	¾	28	0.39	2.21	8.17	14,100	781	36,200	Concrete split.
3	Ransome	¾	28	0.56	3.00	8.32	14,210	555	25,400	Concrete split.
3	Truscon	¾	28	0.44	2.36	8.18	10,290	534	23,400	Rod slipped.
3	Round	¾	28	0.44	2.36	8.01	6,860	363	15,600	Rod slipped.
3	Round	⅜	28	0.11	1.18	7.91	2,950	316	26,800	Rod slipped.
3	Flat	1¼x⅜	28	0.47	3.25	8.06	6,480	247	13,800	Rod slipped.
3	Flat	2x¼	28	0.50	4.50	8.00	9,400	260	18,800	Rod slipped.


STEEL-CONCRETE BEAM TESTS
JOHNSON CORRUGATED BARS
BEAMS
6 UNIVERSITY OF ILLINOIS TESTS + CONCRETE 1·3·6
18 ROSE POLYTECHNIC INSTITUTE " • " 1·3·6
6 UNIVERSITY OF PENNSYLVANIA " ● " 1·2·4
4 UNIVERSITY OF WISCONSIN " ✖ " 1·2·4
6 C.M. & ST. P. RY. " ▲ " 1·2·4
32 BOSTON, TRANS. COMMISSION " ■ " 1·2·3
TOT 72
THEORETICAL POINT FOR JOHNSONS FORMULA FOR 1·2·4 ROCK CONCRETE
THEORETICAL POINT OF JOHNSON'S FORMULA FOR 1·3·6 ROCK CONCRETE
PLAIN CONCRETE BEAM
VALUES OF $\frac{M}{bd^2}$
PERCENTAGE OF REINFORCING
000
100
200
300
400
500
600
0.5
1.0
1.5
2.0



Tests on Full Sized Beams by Prof. Howe at Rose Polytechnic Institute.
Rock Concrete, 1:2:5; Age 115 days. Depth, 14½″; Width, 12″; Span, 15′; Three ¾″ corrugated bars=.93□″.
Theoretical, Mo=625,000″ pounds; Actual, M=655,000″ pounds. Four vertical bars at each end.



Tests on Full Sized Beams by Prof. Howe at Rose Polytechnic Institute.

Rock Concrete, 1:2:5; Age 73 days. Depth, 14″; Width, 12″; Span, 15′; Six ½″ corrugated bars=1.02□″. Theoretical, Mo=725,000″ pounds; Actual, M=929,700″ pounds. Each of the three pairs of horizontal rods bent up vertically at different subdivisions of span.

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GREAT NORTHERN RAILWAY,	ST. PAUL, MINN.
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